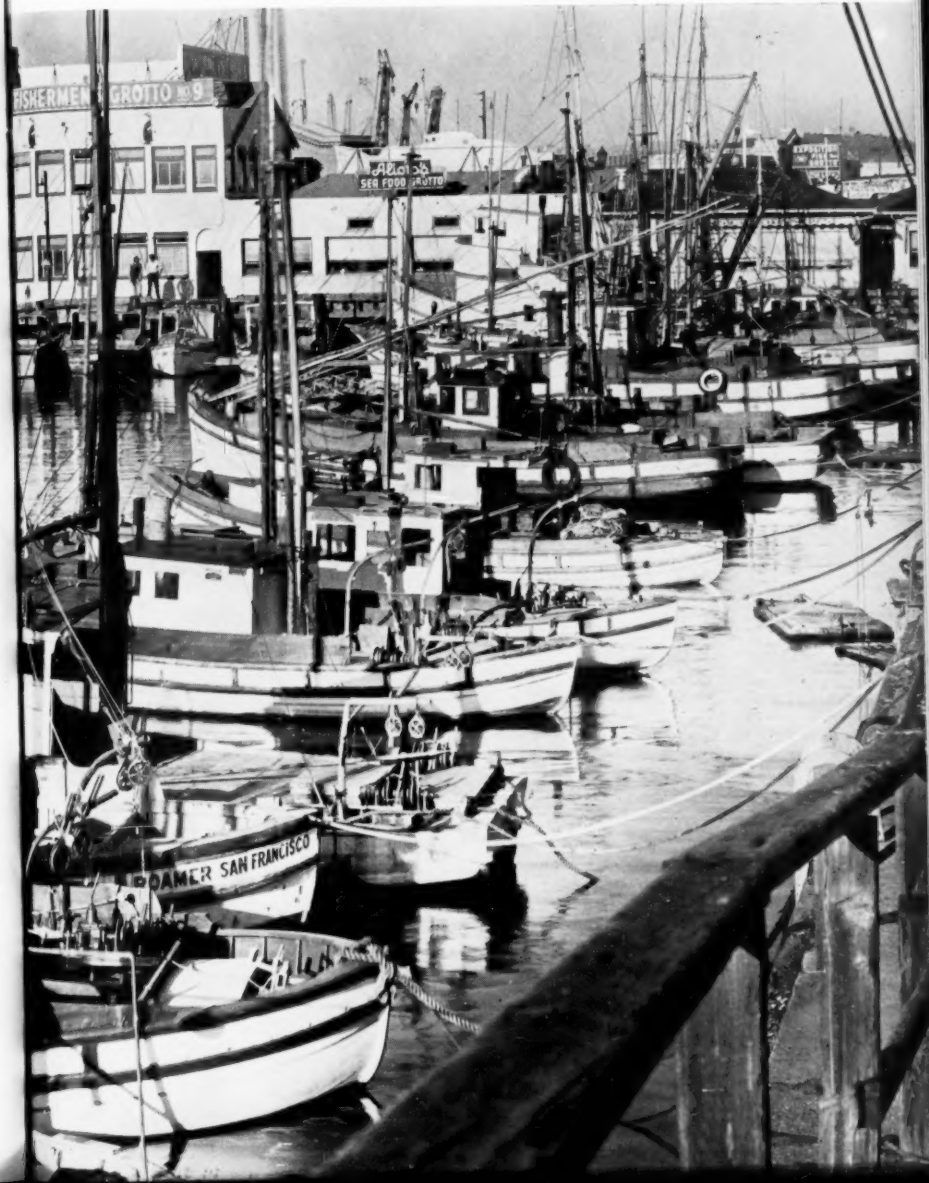
ACI News Letter

Vol. 22 No. 5 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1951



PREPAKT REPORTS ON 1950 OPERATIONS:

279 PROJECTS—ALL TYPES & SIZES!

RAILROAD AND HIGHWAY BRIDGES — 125

Piers and abutments for new bridges; underpinning, stabilization, restoring and encasing of foundations; replacement, and strengthening of piers.

DAMS AND LOCKS — 40

Refacing, dam and foundation stabilization, additions, cut-off walls, cofferdams, and construction facilities.

TUNNELS — 10

Relining, restoring and grouting of lined and unlined tunnels by Intrusion-Prepakt methods—traffic maintained.

FOUNDATIONS AND PILES — 61

Construction, replacement and stabilization of foundations for heavy equipment, public buildings, industrial plants, residential units, power plants and power line towers, grain elevators, and stabilization of highway subsoil.

OTHER INTRUSION-PREPAKT INSTALLATIONS — 29

Tank door frames, railroad facilities and buildings, encasing timber piles, sealing wells, plant construction and stabilization projects.

OTHER SERVICES executed were soil sampling, core drilling, underwater inspections, and government research programs. — 14

OF THE PROJECTS SHOWN 84% ARE REPEAT CUSTOMERS

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Courtesy San Francisco Convention and Tourist Bureau

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1950 ACI Awards

Whitney, Leslie, Cheesman, Colby and Thomson Honored

Charles S. Whitney, J. R. Leslie and W. J. Cheesman, W. S. Colby and H. F. Thomson will receive ACI Awards from President Jackson at the 47th Annual Convention luncheon at the St. Francis Hotel, San Francisco, February 21. These awards by the ACI Board of Direction are on the recommendations of the Awards Committee.

Charles S. Whitney is the choice for the Alfred E. Lindau Award. The award was founded in 1947 by the Concrete Reinforcing Steel Institute in honor of the late Alfred E. Lindau, past president of ACI. It is given only for outstanding contributions to reinforced concrete design practice and is not mandatory each year. The first awardee was Frank E. Richart who received the plaque last year at the Chicago convention.

To W. S. Colby will go the Construction Practice Award for his paper

Design and Construction of a Circulating Water Intake

which was published in the March, 1950, ACI JOURNAL. The award was established in 1944 by the American Concrete Institute to recognize the man on the job for his resourcefulness in translating design into the completed structure.

J. R. Leslie and W. J. Cheesman will receive the Wason Medal for "noteworthy research" reported in

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures

in the September, 1949, ACI JOURNAL. The award, established in 1917 by the late Leonard C. Wason, past president of ACI, is given for original research work on a subject relevant to Institute activity and which within the year has been the subject of a JOURNAL paper.

Continued on p. 4

H. F. Thomson will receive the Wason Medal for "the most meritorious paper" of the year,

Specifications Should Be Realistic

which appeared in the November, 1949, ACI JOURNAL. This award, established at the same time as the Wason Medal for research, is made for the "most meritorious paper" published in the latest volume of *Proceedings*.

The Medalists

Charles S. Whitney—Alfred E. Lindau Awardee

An ACI Member since 1920, Charles S. Whitney long has been active in the concrete field as well as Institute activities.

He was awarded the Wason Medal for most meritorious paper for his 1932 contribution "Plain and Reinforced Concrete Arches." Twice awarded the Fuertes Graduate Medal by Cornell University, he also has been honored with the J. James R. Croes Medal by ASCE. The firm of Ammann and Whitney, consulting engineers of New York and Milwaukee, also received the first Annual Award of the Concrete Reinforcing Steel Institute in 1949 for their work on thin shell concrete structures.

Mr. Whitney is a member of ACI's Board of Direction; chairman of Committee 312, Plain and Reinforced Concrete Arches; chairman of Committee 326, Shear and Diagonal Tension; and a member of Committee 214, Evaluation of Results of Compression Tests of Field Concrete.

As a member of the partnership of Ammann and Whitney, he has been engaged in the design of buildings, airport facilities and bridges, many of these being concrete structures.

Mr. Whitney received a CE degree from Cornell University in 1914 and his MCE in 1915. Following graduation he held positions with Gustav Lindenthal; John Parkinson, architect; A. C. Eschweiler, architect; and Hool and Johnson, engineers. From 1922 to date he has been a consulting and designing engineer.

W. S. Colby—Construction Practice Award

Drawing on his experience with Stone and Webster Engineering Corp., Boston, Mass., W. S. Colby described construction procedures for a circulating water intake for a power plant in the paper which was selected for the Construction Practice Award.

Starting in 1924 as a tracer with Stone and Webster, he subsequently became draftsman, checker, engineer in the structural division and structural engineer. He has also worked for short intervals as structural engineer for McClintock and Craig, Springfield, Mass., United Engineers and Constructors, Procurement Division of U. S. Treasury Dept., U. S. Army Engineers on the Passamaquoddy Project, Eastman Kodak Co., and TVA.

A 1925 graduate of Massachusetts Institute of Technology with a BS in civil engineering, Mr. Colby is a member of the Boston Society of Civil Engineers, ASCE, and the Massachusetts and National Professional Engineers Societies.

J. R. Leslie and W. J. Cheesman—Wason Medalists

J. R. Leslie and W. J. Cheesman have been actively engaged in work on ultrasonic testing for the Hydro-Electric Power Commission of Ontario, Canada, for several years. Mr. Leslie is Research Engineer, Research Division of the Hydro-Electric Power Commission of Ontario and Mr. Cheesman is Assistant Research Engineer in the same organization.

Mr. Leslie graduated from the University of Toronto in 1938 with an engineering physics degree. His ultrasonic experience stems from war work as a civilian scientist with the National Research Council, Ottawa, Canada, and the Minesweeping Section of the Admiralty, London, England. A member of AIEE, he has published several papers on electronics and physics in AIEE publications.

Mr. Cheesman has worked on the development and application of the "Soniscope" since early in 1947. He is a graduate of the University of Toronto in engineering physics and served as a radar officer with the Royal Navy prior to joining the Hydro-Electric research staff.

Continued on p. 25

Program preview, 47th Annual ACI Convention, St. Francis Hotel, San Francisco

February 20-22, 1951

TUESDAY, FEBRUARY 20

9 a.m.—Mezzanine—Convention registration begins—Members and nonmembers—registration fee $2.00

Technical committee meetings as scheduled

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10 a.m.—Colonial Ballroom—Convention called to order by President Frank H. Jackson

STANDARDS—Frank H. Jackson, Session Leader

(ACI 318-47) *Building Code Requirements for Reinforced Concrete*—Committee 318, Frank Kerekes, chairman (Proposed changes appeared in December JOURNAL)

SYNOPSIS—Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special anchorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

(ACI 315-48) *Manual of Standard Practice for Detailing Reinforced Concrete Structures*—Committee 315, Raymond C. Reese, chairman (Proposed changes appear in the January JOURNAL)

SYNOPSIS—Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

(ACI 617-44) *Specifications for Concrete Pavements and Bases*—Committee 617, H. F. Clemmer, chairman (Proposed revisions appeared in October JOURNAL)

SYNOPSIS—New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

Recommended Practice for the Application of Mortar by Pneumatic Pressure—Committee 805, W. L. Chadwick, chairman (Proposed standard appeared in November JOURNAL)

SYNOPSIS—This proposed standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

GENERAL

Factors Influencing Concrete Strength—W. H. Price, U. S. Bureau of Reclamation, Denver, Colo.

SYNOPSIS—The effects of mix proportions, type and brand of cement, availability of moisture for curing, accelerators and curing temperatures on the rate and potential strength development of concrete are discussed. The influence of rate and frequency of load application, dimensions of test specimens and lateral restraint on the indicated strength are also discussed, and information is furnished on the variations in strength which might be expected on a typical job. Compressive, tensile, flexural, bond and shearing strengths are compared, and the strengths of control cylinders are compared with the strengths of cores drilled from structures at later ages. Information is also furnished on loss of strength resulting from freezing and thawing and alkali-aggregate expansion.

2:30 p.m.—Colonial Ballroom—Harmer E. Davis, chairman

DESIGN

Analysis of Earthquake Effects—R. R. Martel, California Institute of Technology, Pasadena, Calif.

SYNOPSIS—Qualitative rules for earthquake resistant design of structures may be drawn from observation of earthquake damage. Variables of building characteristics and combinations of forces acting on them permit different interpretation of data. Acceleration of ground and deflections of structures are important factors. Methods of determining the response of single degree of freedom systems to earthquake-like base motion are considered although in tests the resonators have not been damped as is the case with most structures. These tests and observed damage imply that the seismic coefficient should be smaller for tall buildings than for short ones. The effect of damping, influence of foundation condition and the interaction of adjacent buildings through ground couple require further study.

Design of Earthquake Resistant Structures by Comprehensive Analysis—Charles S. Whitney, Boyd G. Anderson and Mario Salvadori, Ammann and Whitney, New York, N. Y.

SYNOPSIS — Numerical and analytical methods of analysis for the investigation of structures subject to dynamic displacements of the foundation are presented. The numerical methods consist of step-by-step solutions of the equations of dynamic equilibrium by application of finite differences. The analytical solution is obtained by the rigorous integration of the differential equations of motion. These methods are applicable to the investigation of structures having full range of height to width and any number of stories. They allow the inclusion of effects of structural and foundation damping and foundation elasticity.

As an example, the methods are applied to the vibration analysis of a 5-story structure under the action of a sinusoidal half-wave displacement pulse and the displacements experienced at El Centro, Calif., during the Imperial Valley Earthquake of May 18, 1940. The results are compared with current code requirements.

Multistory Buildings Designed to Resist Earthquakes—John J. Gould, San Francisco, Calif.

SYNOPSIS—The objectives of the owner, architect and engineer in the design of reinforced concrete apartment buildings to resist earthquakes are discussed. The design followed the September 1950 recommendations of the Atomic Energy Commission for resistance to atomic blast. The general framing scheme is outlined and the earthquake resistant design features are shown. Foundations and cracking of concrete were major construction problems.

Prestressed Concrete Highway Bridges—Stewart Mitchell, California Department of Public Works, Sacramento, Calif.

SYNOPSIS—To study the use of prestressed concrete for highway bridges a pedestrian bridge over Arroyo Seco was designed and placed under contract. In the United States principal savings are expected to be for formwork rather than in reduction of steel and concrete. Cast-in-place and precast prestressed bridges applicable to California sites are described.

Lack of technical data, cost information and experience restrict the use of prestressing techniques at present. Further studies and better specifications are needed to promote the use of prestressed concrete.

Prevention of Joint Trouble in Concrete Pavements—F. N. Hveem, California Department of Public Works, Sacramento, Calif.

SYNOPSIS—An investigation of joint troubles and failure and some data from a continuously reinforced experimental pavement constructed in 1949 are discussed. Evidence of warping and curling of pavements and the sequence of events leading to pumping and subsequent faulting of the joints are considered.

WEDNESDAY, FEBRUARY 21

Concurrent Sessions will be held morning and afternoon in the **Colonial Ballroom** and **Green Room**

9 a.m.—Green Room—Stanton Walker, chairman

DESIGN

Getting More for Our Concrete Dollar—I. E. Morris, Atlanta, Ga. (This paper appears in the January JOURNAL)

SYNOPSIS—A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. The system is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. The members are designed to use both concrete and steel to maximum advantage.

Spaced and Tied Reinforcing Bar Splices—William T. Walker, U. S. Bureau of Reclamation, Denver, Colo. (The paper appears in the January JOURNAL)

SYNOPSIS—Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little or no advantage would be obtained by spacing deformed reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which lugs could interlock there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Effect of Lateral Loads on Arches—James P. Michalos, Iowa State College, Ames, Ia. (See January JOURNAL for complete paper)

SYNOPSIS—The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shear in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced, with struts normal to the ribs. Numerical studies are included.

Design and Construction of Flat Plate Parking Garages—L. G. Farrant and W. C. Harry, Miami, Fla.

SYNOPSIS—The ideal balance between function, economy and esthetic value has been sought in the construction of flat plate parking structures, culminating in the Blaise garage in New Orleans. Its conception, design and construction were completed with these objectives in mind.

Coordination of the aims was accomplished by sound structural planning and full utilization of the principle of unit buildings—the arrangement of identical parts so that the completed structure is composed of a repetition of the same units, like a set of toy blocks or in this case a set of identical tables resting on each other.

9 a.m.—Colonial Ballroom—Vice-President A. T. Goldbeck, chairman

Soniscope Tests Concrete Structures—E. A. Whitehurst, Purdue University, Lafayette, Ind.

SYNOPSIS—The Soniscope, an instrument which can measure group velocities through as much as 50 ft of concrete, was used for the field testing of concrete structures. Thirteen bridges, one navigation lock, 14 dams and five highway pavements in 12 states were tested. Repeated tests permitted study of changes in the condition of the concrete and the development of wave velocity groups indicating the condition of a structure. The value of the results increases with knowledge of the materials, mix design, method of placement and other characteristics of the structure being tested.

Coarse-Ground Cement Makes More Durable Concrete—H. W. Brewer and R. W. Burrows, U. S. Bureau of Reclamation, Denver, Colo. (See January JOURNAL for complete paper)

SYNOPSIS—Test procedures for measuring the cracking resistance of neat cement, mortar and concrete are described. A relationship between shrinkage cracking and resistance to freezing and thawing is shown. These laboratory tests indicate that coarse-ground cement produces more durable concrete than fine-ground cement.

Entrained Air Simplifies Winter Curing—J. J. Shideler, H. W. Brewer and Wilbur Chamberlin, U. S. Bureau of Reclamation, Denver, Colo.

SYNOPSIS—The investigation described was undertaken to determine the amount of winter protection required to prevent damage to air-entraining concrete by freezing temperatures. A large number of cylinders made with Type II and V cement and various percentages of calcium chloride were cured at temperatures ranging from 10 to 70 F and tested for strength at ages ranging from 3 to 180 days. The resistance of these concretes to accelerated freezing and thawing were compared. The results indicate that the amount of winter protection, as presently specified by the Bureau of Reclamation, can be reduced when air-entraining concrete is used.

False Set in Portland Cement—R. F. Blanks and J. L. Gilliland, U. S. Bureau of Reclamation, Denver, Colo.

SYNOPSIS—False set of cement causes difficulties in mixing and placing and even though the stiffening is eliminated by job conditions, extra mixing or the addition of corrective admixtures, undesirable effects on the hardened concrete remain. It adversely affects water requirement, strength, bond between aggregate and matrix, brittleness and cracking, resistance to freezing and thawing and air-entraining characteristics.

Correctives in the manufacture of the cement are proper cooling of mills or the use of stable calcium sulfate.

Alkali-Aggregate Expansion Inhibitors—W. J. McCoy and A. G. Caldwell, Lehigh Portland Cement Co., Allentown, Pa.

SYNOPSIS—Recent investigators of alkali-aggregate reaction suggest low-alkali cement or substitution of 20 to 30 percent pozzolan for cement as corrective measures.

Experimental data are reported here which indicate that small amounts (1 percent or less) of specific salts added to high alkali cement are effective in reducing expansion. Other tests showed that 0.2 percent or less of selected proteins appear to inhibit expansion to a greater degree than comparable air entrainment induced by conventional air-entraining agents.

LUNCHEON

12:15 p.m. — Colonial Ballroom — President Jackson presents ACI awards

Report of Annual Election and Introduction of New Officers

Retiring President's Address

2:30 p.m.—Green Room—President Frank H. Jackson, chairman

CONSTRUCTION

Finishing and Curing: Key to Durable Concrete Surfaces—Myron A. Swayze, Lone Star Cement Corp., New York, N. Y. (See December JOURNAL for complete paper)

SYNOPSIS—After a comparison of past and present pavement curing and finishing techniques the significance of timing and

Proof...

Dorena Dam
U S Corps of Engineers
Dorena, Oregon
Proof ..that you use less Protex!

Walla Walla Dist. Corps of Eng. Photo
McNary Dam
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Angostura Dam
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Protex meets the requirements of Federal Specifications SS-C-192, ASTM Spec. C-175-48T.

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Name.....

Address.....

City.....*State*.....

character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

Resurfacing Concrete Pavements—H. Walter Hughes, Rochester Department of Public Works, Rochester, N. Y.

SYNOPSIS—A study of background information on the bonding of new concrete to old preceded laboratory bonding tests on wet and dry surfaces. Favorable results led to experiments on topping concrete road slabs which required special techniques in bonding, screeding and finishing. Cost and performance data are presented.

Architectural Concrete—Michael Goodman, University of California, Berkeley, Calif.

SYNOPSIS — Design and construction problems in architectural concrete will be presented. Professor Goodman will draw on his experiences and discuss unusual designs of interest to engineers and architects.

Economy of Lightweight Monolithic Concrete Construction—J. A. Murlin, George L. Dahl, Architects and Engineers, Dallas, Texas

SYNOPSIS—With the defense program threatening to make structural steel increasingly hard to obtain, and with increasing labor costs, any method of construction that promises to effect economies of a dollar per sq ft is of vital interest to architects, engineers and prospective owners.

From accurate construction costs taken from the contractor's records on a cost plus fixed fee contract, it is demonstrated that the use of lightweight expanded shale concrete plus the flat-plate design reduced the cost of a two-story office building by about $0.75 per sq ft for the structural frame. The costs of the building designed for tin-pan joists and beams, for beam and slab and for flat slab using drop panels and splayed columns are compared. The method of analysis used in designing and lightweight flat plate is discussed—bringing to view some improvements in earlier flat plate analysis methods.

Other economies were effected in the plumbing, heating and air conditioning which can only be estimated roughly. Problems encountered in the proper mix design of the lightweight concrete are discussed and mention is made of proper on-the-site mixing methods.

Aggregate Cooling Plant for Mass Concrete—H. H. Roberts, Consolidated Builders, Inc., Mill City, Ore.

SYNOPSIS—To maintain proper temperatures in the 1,500,000 cu yd of concrete in Detroit Dam several techniques were considered. After an investigation of the heat balance in the materials it was decided to construct a plant for cooling the aggregate and cement. The coarse aggregate is immersed in water at 35 F until it is cooled to 38 F. It is then drained and screened to remove excess moisture.

Sand and cement are cooled in continuous hollow-flight screw conveyors in which 35 F water is circulated. Heat transfer is through the conveyor surfaces; there is no direct contact with the cooled water.

2:30 p.m.—Colonial Ballroom—Vice-President H. F. Thomson, chairman

PRECAST CONCRETE

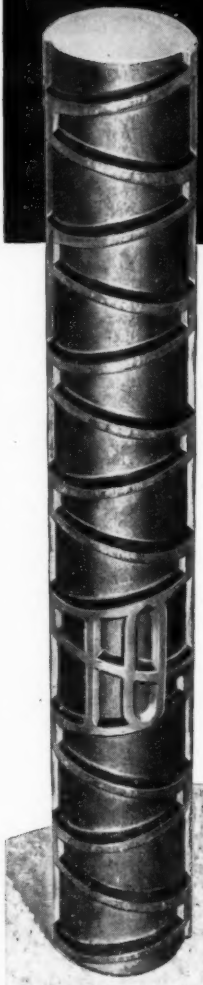
Advances in Precast Floor Systems—F.N. Menefee, University of Michigan, Ann Arbor, Mich.

SYNOPSIS—Reinforced concrete floors and roofs for office and school buildings, apartments and similarly loaded structures are being made of precast block assembled to form a joist or slab; and also of precast joists with precast filler blocks.

The high cost of carpentry has brought about a successful attempt to eliminate most of the previously required formwork. The trend is toward units made of high strength, lightweight aggregate under closely controlled factory conditions.

New machinery has been developed for the production of these new shapes and several floor systems have been worked out by block manufacturers. Those described in detail are the Dox floor and roof system, the F and A system, Joistile and Strestcrete. Principal characteristics and uses of each are discussed.

USE THIS *New-Billet Bar* FOR FIRM ANCHORAGE!



The size of each Bethlehem bar is easily determined by means of numeral rolled into the bar.

The improved Bethlehem Reinforcing Bar—that's the bar to specify for firm anchorage in concrete!

It has specially designed lugs which meet the deformation requirements of ASTM Specification A305. The lugs are alternately sloped and uniformly spaced. Their gripping action in the concrete means less slips at working loads—maximum bond strength—firm anchorage. Thus wide tensile cracks in the concrete are less likely to occur.

The improved Bethlehem Bar is made from new-billet steel which meets ASTM Specification A15. Conforming to Simplified Practice Recommendation R26-50, the weight of each bar is designated by number. There is no change in either the standard unit weights or nominal cross-sectional areas—hence there is no need to alter basic design practices. All sizes of the Bethlehem Bar are rolled as round sections. The nominal round diameters, plus the new and old size designations, are shown in the table below.

Send for copy of our reinforcing bar descriptive sheet.

New Standard Size Markings of Bethlehem Reinforcing Bars

New No.	3	4	5	6	7	8	9	10	11
Nominal Round Diameter	.375	.500	.625	.750	.875	1.000	1.128	1.270	1.410
Old Size Designation, Inches	3/8 ○	1/2 ○	5/8 ○	3/4 ○	7/8 ○	1 ○	1 □	1 1/8 □	1 1/4 □

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

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BETHLEHEM STEEL

BETHLEHEM REINFORCING BARS

Precast Structural Elements—A. G. Streblow, Basalt Rock Co., Napa, Calif.

SYNOPSIS—Experience with precast piles, joists and cored roof and floor sections and the introduction of automatic block making machines suggested the development of new precast products. The considerations influencing and determining the design of Stresteerete elements are discussed in detail.

The characteristics and application of floor and roof slabs, wall sections, prefabricated packaged buildings, columns, beams, planks and special structures are described.

Precast Concrete Construction—Otto Safir, Marwell Construction Co. Ltd., Vancouver, B. C., Canada

SYNOPSIS—Beams and columns for the precast concrete frames of warehouses were cast flat in multiple forms on the ground slab. Cold weather required special precautions in mixing, placing and curing the concrete. Erection was by crane and the members were fastened in place by bolts.

In a parking garage precast elements were combined with cast-in-place members to form an essentially monolithic structure. The job was complicated by the necessity for minimum interference with parking of vehicles.

A 25-ft high retaining wall was constructed of precast counterforts and 8-in. wall planks and erected on cast-in-place foundations. Principal features of the job were the complicated shape of the counterforts and the weight of the completed units.

Precast Concrete as Protection Against Atomic Blast—A. Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

SYNOPSIS—Precast concrete presents an excellent means for providing protection against atomic blast. The technique is suitable for use in new construction as well as adaptable to existing structures by providing them a protective shell. The needed material for this purpose will consist of readily-assembled framing elements which can be prefabricated at regional plants and stored or stock-piled at various districts for immediate use in an emergency. The precast framing elements will comprise cells and ribbed panels of various shapes and outline. The paper contains suggested arrangements and details of assembly of those elements to form buildings and shelters. A discussion is also given of the relative merit of each type, together with an analysis of anticipated resistance under blast pressures.

Tilt-Up Construction—F. Thomas Collins, San Gabriel, Calif.

SYNOPSIS—Early design and construction methods and more recent developments affecting design are reviewed briefly. The patent situation is considered and found not restrictive to the use of tilt-up construction.

Design suggestions are given for poured wall-columns, joints and seals, wall splices, anchorage, and bearing and nonbearing walls. Wall, bent and roof slab erection is described. Site fabrication of parts is recommended and flat slab technique is emphasized as the most convenient, cleanest, and most satisfactory method.

Tilt-up construction is offered as a means of saving steel, lumber and cement and its use by governmental agencies is urged.

Prestressed Concrete—J. R. Janney, Portland Cement Assn., Chicago, Ill.

A sound movie in color will show the formwork, placing of prestressing strands, placing and curing of concrete, prestressing and testing of the completed beam.

Pertinent test data will be reviewed and a summary of conclusions will be given regarding future studies planned on prestressed concrete.

THURSDAY, FEBRUARY 22

9 a.m. — Colonial Ballroom — Research — an open session of Committee 115, Research—S. J. Chamberlin, chairman; George W. Washa, secretary

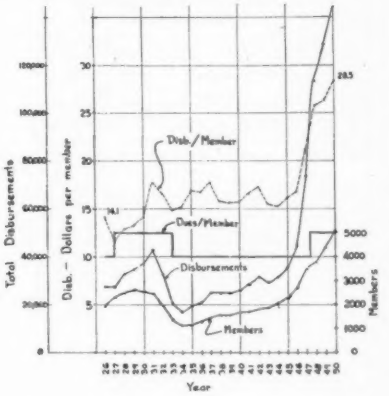
A group of short papers on research techniques and preliminary results on prestressed concrete, reinforcement, factors affecting resistance to freezing and thawing, air entrainment and other subjects. Papers not for publication or press release.

2 p.m.—Colonial Ballroom—Panel discussion—based on questions submitted by ACI Members. C. H. Scholer, Kansas State College, presiding

Final selection of panel members still to be made but on the basis of last year's experience a lively and interesting exchange of opinions may be expected.

By-Laws Changes

On p. 14 of this News Letter is formal notice of proposed By-Laws changes to be considered for adoption at the annual convention in San Francisco, February 20, 1951. Important are changes to increase dues rates of membership classes except Junior and Student. These are a reluctant recommendation of the Board of Direction that the Institute may continue its functions without undue loss of service to members.



Membership has increased 250 percent while costs have increased 550 percent

It is trite to say that costs of almost everything have increased greatly in the last few years. The chart on this page shows graphically a comparison of expenditures versus number of members as well as the expenditure per Institute member. The expenditure per Institute member has been well above the annual dues rate of an individual member, the difference being made up largely by revenue from publications sales.

Nor has all increase in expenditures resulted wholly from increased unit costs. A more nearly adequate staff provides greatly increased service in coverage and editorial quality of the JOURNAL. Since 1946, the JOURNAL has returned to its schedule of ten issues per year abandoned in depression years. Papers and current reviews published have increased sharply in the last two years with a proportional rise in the number of JOURNAL pages.

In proposed By-Laws changes it is intended that the Institute raise the high standard of committee and editorial activity and the JOURNAL as our basic means of service.

Harry Diamond

Proposed Changes in ACI By-Laws

Notice is hereby given to the membership of the American Concrete Institute that at the Institute's 47th Annual Convention, San Francisco, February 20-22, 1951, consideration will be given to a petition for By-Laws Amendments.

The undersigned members of the American Concrete Institute hereby petition the Board of Direction to submit to the membership for adoption, in accordance with Article VI of the By-Laws, the following proposed changes in the By-Laws. (The left-hand column repeats the present wording of the By-Laws; the right-hand column the proposed rewording. Omitted unchanged sections are indicated by ellipses):

ARTICLE I—MEMBERS

PRESENT	PROPOSED
Section 1. This Institute shall consist of Members, Contributing Members, Junior Members, Student Members and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.	*Section 1.* This Institute shall consist of Members, Corporation Members, Contributing Members, Junior Members, Student Members and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.
Sec. 2. A Member or a Contributing Member may be a person, firm, corporation, society or other organization. (No age limit is specified for Members; however, for the benefit of the younger member applicants, two optional grades, Junior Member and Student Member, are provided.) A Junior Member shall be a person not more than 28 years of age. A Student Member shall be a student in residence at a recognized technical or engineering school. An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.	*Sec. 2.* A Member shall be an individual. A Corporation Member shall be a firm, corporation, society, agency of government or other organization. A Corporation Member shall name one individual as its representative who will enjoy all membership rights and privileges. A Contributing Member shall be an individual, firm, corporation, society, agency of government or other organization wishing to give larger support to Institute activities through the payment of larger dues. Contributing Members, other than individuals, shall name representatives as do Corporation Members. A Junior Member shall be a person less than 28 years old. A Student Member shall be less than 28 years old and a student in residence at a recognized technical or engineering school. An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.
Sec. 3. A Member, Contributing Member, or Junior Member shall be proposed by two Members of the Institute and elected by a two-thirds vote of the Board of Direction. A Student Member shall be proposed by a member of the faculty of his institution and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be nominated by ten members of the Institute and elected by unanimous vote of the Board of Direction in secret ballot.	*Sec. 3.* All classes of Members, except Honorary Members and Student Members, shall be proposed by at least one Member of the Institute and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be proposed by a member of his school's faculty, who need not be a member of the Institute, and elected by a two thirds vote of the Board of Direction.
* * *	* * *
Sec. 5. A firm, corporation, society or other organization holding membership may name one individual as its representative who shall enjoy all membership rights and privileges.	*Sec. 5.* (Delete)
Sec. 6. Applications for membership on forms prescribed by the Board of Direction, and resignations from membership, shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from members whose dues are not more than sixty days in arrears, except by special action of the Board of Direction.	*Change Sec. 6 to Sec. 5.* Applications for membership, resignations from Members and requests for change of representatives of Corporation or Contributing Memberships shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

ARTICLE II—OFFICERS

* * *

Sec. 3. Before September 15 of each year the Committee on Nominations shall, by letter ballot of its members, nominate candidates for offices to become vacant at the next annual convention and twenty candidates for membership on the Committee on Nominations which is to serve in the following year, and shall transmit the names of all candidates thus nominated to the Secretary-Treasurer of the Institute. In the selection of candidates for Directors, the Committee on Nominations shall have due regard for diversity of professional and geographical representation. The consent of each candidate for office must be obtained before notice of his nomination is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. Upon petition to the Board of Direction signed by at least ten members of the Institute, additional nominations for offices or for membership on the Committee on Nominations may be made within 30 days thereafter.

The complete list of nominations shall be submitted 60 days before the next convention to the Institute membership for letter ballot to be canvassed at 5 p.m. on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be chairman of that committee; the four next highest shall be declared elected members of the committee. With these five the three past-president members of the Board of Direction shall serve, making a total membership of eight.

Should any member of the Committee on Nominations thus chosen fail, within fifteen days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elected places on the committee shall be filled.

Sec. 4. The terms of office of the President and the Secretary-Treasurer shall be one year. At the first election after the ratification of these amended By-Laws, one Vice President shall be elected for a term of one year and one for a term of two years; thereafter one each year for a term of two years; at the first election after the ratification of these By-Laws, three Directors shall be elected for terms of one year, three for terms of two years and four for terms of three years; thereafter, four each year for terms of three years. A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive annual conventions.

* * *

Sec. 8. Vacancy in any office, for the unexpired term, shall be filled by appointment by the Board of Direction except as povided in Section 7.

Sec. 9. In the event of disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have the power to declare the office vacant.

* * *

Sec. 3. Before September 1 of each year the Committee on Nominations shall report to the Secretary-Treasurer of the Institute the candidates nominated for offices to become vacant at the next annual convention and twenty candidates for membership on the Committee on Nominations which is to serve in the following year. In the selection of candidates for Directors, the Committee on Nominations shall have due regard for diversity of professional and geographical representation. Each candidate for Board membership must have given consent to his nomination before the report is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. Upon petition to the Board of Direction signed by at least ten members of the Institute, additional nominations for offices or for membership on the Committee on Nominations may be made within 30 days thereafter.

The complete list of nominations shall be submitted 60 days before the next convention to the Institute membership for letter ballot to be canvassed at 5 p.m. on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be chairman of that committee; the four next highest shall be declared elected members of the committee. With these five the three past-president members of the Board of Direction shall serve, making a total membership of eight.

Should any member of the Committee on Nominations thus chosen fail, within fifteen days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elected places on the committee shall be filled.

Sec. 4. Terms of office shall be as follows: President, one year; Secretary-Treasurer, one year; Vice-President, two years, with one Vice-President elected alternate years; Directors, three years, with four Directors elected each year. A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive annual conventions.

* * *

Sec. 8. Vacancy in any office shall be filled by appointment by the Board of Direction for the unexpired term, except as provided in Section 7.

Sec. 9. After the word "shall" delete the words "have the power to."

ARTICLE IV—DUES

Section 1. Dues for the several membership classes shall be payable annually in advance from the first of the month of notification of the Member applicant of his election by the Board of Direction as follows: Contributing Members, $50.00; Members, individuals, in the Dominion of Canada and in the United States and its territories and possessions, $12.50; elsewhere, $10.00; firms, corporations, societies or other organizations, $25.00; Junior Members, $7.50; Student Members, $5.00; Honorary Members, none. Any individual Member may be admitted to life membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

* * *

Sec. 3. A Member whose dues remain unpaid for a period of six months shall forfeit the privileges of membership and shall be officially notified to this effect by the Secretary-Treasurer. If these dues are not paid within six months thereafter his name shall be stricken from the list of Members, unless otherwise specifically ordered by the Board of Direction. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

Section 1. Dues shall be payable in advance on the first day of the month of notification of the Member applicant of his election by the Board of Direction, and annually thereafter, as follows: Contributing Members, $100.00; Corporation Members, $50.00; Members (individuals) in North America* and U.S. possessions, $15.00; Members elsewhere, $12.00; Junior Members, $7.50; Student Members, $5.00. Corporation Members outside North America may upon application be granted a 20 percent reduction in dues. Any individual Member may be admitted to life membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

* * *

Sec. 3. A Member in any classification whose dues remain unpaid for a period of two months shall forfeit the privileges of membership and shall be sent notice to this effect by the Secretary-Treasurer. A Member in any classification whose dues are unpaid for a period of one year from their due date shall have his name stricken from the rolls unless otherwise specifically ordered by the Board of Direction. Members may be reinstated upon payment of all indebtedness against them on the books of the Institute and dues for one year in advance of that date.

Signed by:

R. F. Blanks	R. W. Spencer
A. E. Cummings	M. A. Swayze
H. C. Delzell	H. F. Thomson
A. T. Goldbeck	I. L. Tyler
Frank H. Jackson	Stanton Walker
N. M. Newmark	C. E. Wuerpel
H. J. Gilkey	C. S. Whitney
C. H. Scholer	R. B. Young

The foregoing petition proposes changes and clarifications in the By-Laws as follows:

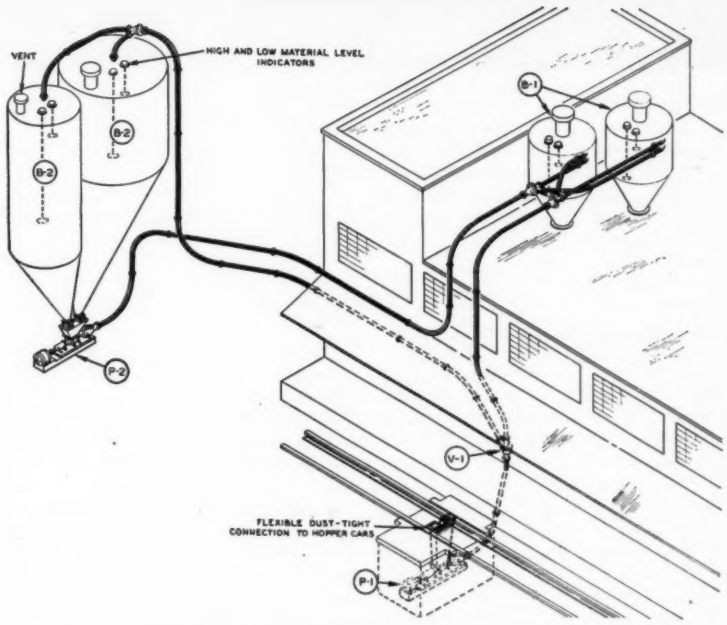
1. In Article I—Members, Corporation Member is added in Section 1. Section 2 defines membership grades more specifically and puts an age limit of 28 on Student Members. Section 3 changes sponsors from two to one and clarifies Student Member sponsorship. Section 5 becomes unnecessary because of revisions in Section 2. Section 6, changed to 5, now specifies that requests for change in Corporation representatives shall be made in writing.

2. In Article II—Officers, Section 3 changes the date of the Nominating Committee report from September 15 to September 1. Section 4 simplifies the description of terms of office. In Section 8 a phrase is transposed. Section 9 changes from permissive to mandatory the action of the Board of Direction in the event of the inability of an officer to perform.

3. In Article IV—Dues, Section 1 changes dues as follows: Contributing Members, from $50.00 to $100.00; Corporation Members, from $25.00 to $50.00; Individual Members in North America* (formerly Individuals in United States and Canada) from $12.50 to $15.00 and Individuals elsewhere, from $10.00 to $12.00. Corporation Members outside North America are given a 20 percent reduction in dues from the new rate. Junior and Student Member rates are unchanged.

Section 3 changes from six months to two months the period of sending Journals to Members whose dues are not paid.

*Includes all countries and territories in Central America and the West Indies. Authorities: *Commercial Atlas of America*, Rand McNally & Co., Chicago; and *Survey Atlas of the World*, The Times, London.



Fuller-Kinyon System conveys either Portland cement or fly ash

The above illustrates how a cement-products plant in the mid-west uses the Fuller-Kinyon Conveying System for handling of two different materials, used in the manufacture of its product.

Portland cement and fly ash are received in hopper-bottom cars, fed by gravity to a Fuller-Kinyon Pump (P-1) in pit underneath railroad track. This pump conveys materials, through the use of a two-way valve (V-1), either direct to process bins (B-1), or to storage bins (B-2). A second Fuller-Kinyon Pump (P-2) conveys materials from bins (B-2) to process bins (B-1). A very simple and flexible system of unloading and conveying, economically possible only with Fuller-Kinyon Pumps.

If costs for conveying have been cutting into your profits . . . if maintenance costs are climbing . . . you'll find it profitable to call in Fuller. Chances are we can show you how minimum expenditure can bring you maximum operating efficiency with consequent improvements in profits.

fuller

FULLER COMPANY, Catasauqua, Pa.
120 S. LaSalle St., Chicago 3
420 Chancery Bldg., San Francisco 4

P-116

Who's Who This Month

Committee 315

ACI Committee 315, Detailing Reinforced Concrete Structures, under the chairmanship of Raymond C. Reese, consulting engineer, Toledo, Ohio, submits "Proposed Revisions to Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)," p. 349.

The committee includes Raymond Archibald, Bureau of Public Roads, Washington, D. C.; Frank H. Beinhauer, consulting engineer, Davenport, Ia.; Everett E. Ebling, Bethlehem Steel Co., Chicago, Ill.; Frank Kerekes, Iowa State College, Ames, Ia.; Henry L. Neve, Bureau of Reclamation, Denver, Colo.; Fred L. Plummer, Hammond Iron Works, Warren, Pa.; E. E. Rippstein, Laclede Steel Co., St. Louis, Mo.; and John F. Seifried, Ceco Steel Products Corp., Chicago, Ill.

The committee, organized in 1936, presents the proposed revisions to the 1948 manual for discussion with a view to their consideration for adoption at the 47th annual convention in San Francisco, Feb. 20-22, 1951.

The proposed revisions change bar designations to conform to the numbered designation of the U. S. Department of Commerce. All drawings are also changed to agree with new bond values and anchorage details for new-style deformed bars.

Harold W. Brewer and Richard W. Burrows

Making their first appearance in JOURNAL pages, Harold W. Brewer and Richard W. Burrows, U. S. Bureau of Reclamation, Denver, Colo., are the authors of "Coarse-Ground Cement Makes More Durable Concrete," p. 353. The paper is scheduled for the convention program in San Francisco.

During the last few years Mr. Brewer and Mr. Burrows, materials engineers in the Research and Geology Division of the Bureau have been attempting to determine the "why" of concrete durability.

Mr. Brewer graduated from Colorado A & M college in 1930 and worked for three years with the Interstate Commerce Commission in Washington, D. C. He has been with the Bureau of Reclamation since 1933 and in charge of freezing and thawing investigations for ten years.

Mr. Burrows joined the Bureau in 1946. A graduate in mining engineering from the Colorado School of Mines, he was previously employed as a metallurgist by the Aluminum Company of America and served as a radar technician in the Navy during World War II.

Marvin L. Mass and Jack R. Janney

Marvin L. Mass, Assistant Professor, University of Colorado, Boulder, Colo., and Jack R. Janney, research engineer, Portland Cement Assn., Skokie, Ill., are the authors of "Simple Equipment Economically Explores Prestressing," p. 361.

Professor Mass has been assistant professor of civil and architectural engineering at the University of Colorado for 2½ years, teaching primarily reinforced concrete design. He initiated the research program on prestressed concrete at the university. Prior to joining the staff at Boulder, Professor Mass was an engineer with the Boeing Aircraft Co., Seattle, Wash.; construction engineer with Northwestern Engineering Co., Denver Colo.; and an instructor at Illinois Institute of Technology, Chicago, Ill. He is a graduate of Purdue University and received an MS from Illinois Institute of Technology.

Mr. Janney is now assistant development engineer at the PCA laboratory in Skokie. He has been working chiefly on research on prestressed concrete, continuing studies begun at the University of Colorado while working for an MS degree.

William T. Walker

"Laboratory Tests of Spaced and Tied Reinforcing Bars" are described on p. 365 by William T. Walker, engineer, Structural Research Section, Bureau of Reclamation, Denver, Colo.

His work at the Bureau is concerned primarily with the testing of reinforced concrete models and the analyzing of test results. He was co-author with Douglas McHenry of

the June 1948 JOURNAL paper "Laboratory Measurements of Stress Distribution in Reinforcing Steel."

Mr. Walker received a civil engineering degree from the University of Tennessee in 1939 and an MS in 1940. He entered government work at the Charleston Navy Yard and later transferred to the Navy Bureau of Ships where he was engaged in the structural design of ship hulls. He joined the Bureau of Reclamation in 1946 where he has been occupied with reinforced concrete research.

Charles W. Allen

Charles W. Allen, research engineer, Ohio Highway Dept., Columbus, Ohio, discusses "Finishing Air-Entraining Concrete Pavements," p. 373. This paper was originally presented at ACI's Washington Regional Meeting in Washington, D. C.

A graduate of Ohio State University, he was an instructor in civil engineering and junior research engineer in the engineering experiment station at the university before joining the plant department of the New York Telephone Co.

Mr. Allen has been with the Ohio Highway Dept. since 1932 in various capacities at the Testing and Research Laboratory. He served as Acting Chief Engineer, 1941-46, and was appointed Research Engineer in 1946.

James P. Michalos

Scheduled for convention presentation in San Francisco in February, "Effects of Lateral Loads on Arches," p. 377, is by James P. Michalos, Associate Professor of Civil Engineering, Iowa State College, Ames, Ia.

Professor Michalos is a civil engineering graduate from the University of Wisconsin and received a master's degree from Yale University and a Ph.D. degree from Northwestern University. He has had extensive experience with consulting firms, industry and TVA in design, analysis and construction of various types of structures. Prior to joining the staff at Iowa State College, he taught at Montana State College and Syracuse University. He is a member of ACI Committee 314, Rigid Frame Bridges.

I. E. Morris

"Getting More for Our Concrete Dollar," p. 389, is a design paper by I. E. Morris, consulting engineer, Atlanta, Ga., and is also scheduled for presentation at ACI's 47th annual convention in San Francisco.

Mr. Morris is a member of Committee 318, Standard Building Code, and the author of "Economy in Structural Design" in the January 1948 JOURNAL.

He has had wide experience in design of concrete structures. For eight years he was chief engineer of the reinforcing steel department of Connors Steel Co. In 1938 he entered private structural practice, and in 1944 organized the firm of I. E. Morris and Associates, engineers and consultants.

Herman Schorer

Herman Schorer, well known for his work with prestressed concrete, died recently at Valhalla, N. Y.

The designer of one type of self-contained prestressed unit, Mr. Schorer had also served as manager and president of Borsari Tank Corp., New York, specializing in design and construction of concrete tanks.

Affiliated with ACI since 1935, Mr. Schorer was a member of Committee 323, Prestressed Reinforced Concrete, and had published two contributions on prestressed concrete in the ACI JOURNAL.

He was born in Switzerland and was graduated from Federal Polytechnic Institute, Zurich, in 1914. He was employed for hydraulic investigations, 1914-17, by the Swiss Bureau of Water Resources and, 1917-20, by the Lonza Power Co. Mr. Schorer came to the United States in 1920 as a draftsman for the New York Central Railroad and Merchants Dispatch Transportation Co. From 1922-23 he was field draftsman and designer, San Joaquin Light and Power Co., Fresno, Calif.; in 1924, concrete designer, Estuary Subway Office, Oakland, Calif. From 1926-32 he was hydraulic engineer and chief engineer for Thebo, Starr and Anderton, San Francisco, Calif. He was connected with the design of the Glines Canyon arch dam and penstock, supervised the design of the Guadalupe River power development, Columbia, S. A., and various other hydro-electric projects. He joined the Borsari Tank Corp. in 1934 as manager and president.

Positions and Projects — ACI Members

Siess appointed chairman

Prof. Chester P. Siess, University of Illinois, Urbana, Ill., has been appointed chairman of Subcommittee 3 of ACI Committee 323, Prestressed Reinforced Concrete. He succeeds the late Herman Schorer. The subcommittee is at present compiling information on notations, definitions, specifications and codes relating to prestressed concrete.

Hirschthal and King open office

M. Hirschthal and Philip King recently organized the firm of Hirschthal and King, Consulting Engineers, New York City. Mr. King was formerly supervising engineer for the firm of Seelye, Stevenson and Value. Mr. Hirschthal entered consulting engineering practice last year after retiring from the engineering staff of the Delaware, Lackawanna and Western Railroad.

Steinman honored

At the annual meeting and dinner of the New York Academy of Sciences in December, D. B. Steinman, noted bridge engineer of New York City, was elected a Fellow of the Academy and Member of the Council of the Academy. Dr. Steinman presented an illustrated lecture on "Romance of Bridges" in which he gave a vivid portrayal of the engineer's creative ability to combine science and art in the building of great and beautiful spans.

Brunnier honored

San Francisco's Building Industry Conference Board has honored Henry J. Brunnier, nationally known consulting engineer working in earthquake resistant construction. He has received the organization's "Award of 1950."

The Building Industry Conference Board represents engineering, contracting, architectural, materials and banking associations of northern California.

Collier named senior engineer

Theodore F. Collier, formerly in charge of structural and architectural layout and design of various steam electric and industrial developments for Sanderson and Porter, has joined the staff of Westcott and Mapes, consulting engineers of New Haven, Conn., as senior structural engineer.

Corporations combine

The assets and business of the Lima-Hamilton Corp. recently became a part of the Baldwin Locomotive Works, which under the name of Baldwin-Lima-Hamilton Corp. will continue to carry on all the activities formerly conducted by both companies.

Short course on aggregates and concrete

Ninety-eight students, representing 63 companies in the sand and gravel and ready-mixed concrete industries, attended the fifth annual short course on aggregates and concrete held recently at the University of Maryland, College Park, Md. The course was sponsored by the University of Maryland, the National Sand and Gravel Assn. and the National Ready Mixed Assn. The course consisted of lectures and demonstrations on basic and fundamental technical information on aggregates and concrete.

New York sets up building code liaison

The New York State Building Code Commission recently established liaison with the agencies in the 36 other states concerned with building regulations on a statewide basis. The purpose of the liaison as explained by George Bain Cummings, vice-chairman of the Commission, is to further a reciprocal arrangement on information on building code matters to advance uniformity and improvement in building laws and regulations in New York and the other states.

"It is our belief that the liaison mechanism being established with other state agencies may be a medium for accelerating code improvement on a nation-wide basis," Mr. Cummings declared. State agencies throughout the country have shown interest in New York's approach to a performance-type code he said.

Courtesy San Francisco Convention and Tourist Bureau

So you're going to San Francisco?

A city of surprises

Whatever your mood, San Francisco will not disappoint you. There is much to see and to do in leisure hours between convention sessions. Here are a few of the things you don't want to miss:

Swing aboard one of the cable cars as it climbs up the steep slope of one of the city's hills. Explore fabled Chinatown—where America suddenly seems to be 6000 miles away. Visit the Oriental telephone exchange where calls are given by the 2400 subscribers' names instead of by numbers.

Don't miss Fisherman's Wharf—a bit of Naples to the eye, a bit of heaven to your nostrils if you enjoy fresh seafood.

Survey the city by night atop Telegraph Hill where you can see both great bridges, Treasure Island, Russian Hill, Nob Hill, even Alcatraz—all of the city spread before your feet.

You shouldn't leave San Francisco without a visit to the Golden Gate Park whose thousand acres were reclaimed from the sand dunes. This, the place where a local newspaper in 1880 said, "a blade of grass cannot be raised without four posts to support it," is now one of the beauty spots of the world. Here can be found the De Young Museum, rare and colorful tropical fish in the Steinhart Aquarium, the remarkable African collection of animals in the adjoining Academy of Sciences Building.

For a quiet visit to the past, stop at Mission Dolores, the church where San Francisco was born.

San Francisco is a fascinating metropolis—a city full of surprises.

New Members

ACI now has another geographical location on its records—a new member from Burma. Canada was in the lead with 6 new members for the month of November—Illinois and Venezuela tied for second with 4 each. New member applicants for November—44—bring the total membership to 5216.

California

Hill, Raymond A., (Indiv.) 1000 Edison Bldg., 601 W. Fifth St., Los Angeles 17, Calif.
Ward, Tom, (Jr.) 2924 26th St., Sacramento, Calif.

Georgia

Bustamante Perez, Luis, (Indiv.) 817 Hickman Rd., Augusta, Ga.

Illinois

Davis, Wilson L., (Indiv.) 6743 S. East End Ave., Chicago 49, Ill.
Elstner, Richard C., (Jr.) 108 Talbot Laboratory, University of Illinois, Urbana, Ill.
Noda, Jim M., (St.) 321-3-C P. G. U., Champaign, Ill.
Worth, Willard J., (Indiv.) 420 Catherine St., Ottawa, Ill.

Indiana

Hedderich, H. F., (Indiv.) 3821 E. New York St., Indianapolis, Ind.
Schwendler, Robert George (St.) 128 Walsh Hall, Notre Dame, Ind.

Louisiana

Strange-Boston, Donald L., (St.) 824 Oneonta St., Shreveport, La.

Massachusetts

Homsy, Norman A., (Indiv.) 72 Easton St., Allston 34, Mass.

Michigan

Shugart, Joe L., (Indiv.) 6065 Cadillac, Detroit 13, Mich.

Mississippi

Maxwell, Clyde V., Jr., (Indiv.) First Federal Savings & Loan Bldg., Jackson, Miss.

Missouri

Coyle, William V., (St.) 301 S. Glenwood Ave., Columbia, Mo.
Fistedis, Stan H., (St.) Civil Engineering Dept., University of Missouri, Columbia, Mo.
Hahn, Robert L., (Indiv.) 1911 Baltimore Ave., Kansas City 8, Mo.

New Mexico

Subt, A. G., (St.) 1606-16 Loop Sandia Base, Albuquerque, N. M.

New York

Murphy, John J., (Indiv.) 445 Franklin St., Buffalo 2, N. Y.

Ohio

Goetz, John L., (Indiv.) Southwestern Portland Cement Co., Fairborn, Ohio
Weigel, J. F., (Indiv.) Medusa Portland Cement Co., 1000 Midland Bldg., Cleveland 15, Ohio

Oregon

Ellsworth, Richard D., (Indiv.) 3124 NE 67th Ave., Portland 13, Ore.
Warren, Sam F., (Indiv.) Box 85, Detroit, Ore.

Wisconsin

Koehnen, Harvey E., (Jr.) 912 N. 29th St., Milwaukee 8, Wis.
Riverside Dunbrik Co., (Corp.) 6122 N. 76th St., Milwaukee 9, Wis. (John A. Krause)
Sherrod, R. T., (Indiv.) 2378 N. 60th St., Milwaukee 10, Wis.

Wyoming

Covington, Chester Lee, (Jr.) 515 S. 12th, Laramie, Wyo.

Puerto Rico

Villamil, Ivan A., (Jr.) c/o Dr. Jose R. Villamil, Villalba, Puerto Rico

Burma

Kayser, Svend, (Indiv.) c/o The East Asiatic Co., Ltd., P. O. Box No. 845, Rangoon, Burma

Canada

Berry, William A., (Indiv.) 129 Coleridge Ave., Toronto 13, Ont., Canada
Busby, A. H. W., (Indiv.) The Consolidated Mining and Smelting Co. of Canada, Ltd., Trail, B. C., Canada
Davis, Charles Gordon, (Indiv.) Apt. No. 9, 4955 Maplewood Ave., Montreal, Que., Canada
Diplock, Frank A., (Indiv.) c/o Diplock Durable Floor Co., Room 505, 57 Bloor St., W., Toronto, Ont., Canada
Royds, James H., (Indiv.) 1529 MacGregor St., Montreal 25, Que., Canada
Sugiyama, J. S., (Indiv.) c/o Rule, Wynn and Rule, Archts., Blow Bldg., Calgary, Alberta, Canada

England

Gallai-Hatchard, M., (Indiv.) 5 Queen's Acre, Cheam, Surrey, England

Guatemala

Enrique Azmitia C., (St.) 13 Calle Pte. 1-C (Apda. Postal 348), Guatemala

D'Arcy, Edgar de Leou (St.) 16 Calle Poniente No. 46, Guatemala City, Guatemala

Keller, Jorge Andrade, (St.) 3° Calle Oriente de Jocotenango No. 8, Guatemala, Guatemala

Mexico

Donde Gorospe, Manuel, (Indiv.) Atbixco 55, Mexico 11, D. F., Mexico

Planta De Concreto Depto. Madereria Matamoros (Corp.), P. O. Box No. 48, H. Matamoros Tamps, Mexico (Mariano Marin, Jr.)

Venezuela

Martin Brito, Jose Noel, (St.) Bloque 6 - Apartamento P - No. 6, Urbanizacion El Silencio, Caracas, Venezuela

Mogna Cruz, Freddy, (St.) Sur 15 - No. 84 - El Conde, Caracas, Venezuela

Ormaechea, Juan Andres, (Indiv.) Oeste 3 - No. 17, Caracas, Venezuela

Yunis, Natalio, (Jr.) Caja de Agua a Luneta - 11, Caracas, Venezuela

Honor Roll

There is still time before the close of this year's Honor Roll to raise your name to one of the top ten. The Morgans are again holding the top two spots with a total of 66½ credits—Leon Venegas is in third place with 10 credits.

Don't forget the current Honor Roll ends January 31, 1951!

Newlin D. Morgan (Ill.)	44½
Newlin D. Morgan, Jr. (Wyo.)	22
Leon Venegas (La.)	10
Rafael Ruiz P. (Guatemala)	9
Henry E. Griset (N. C.)	7
E. I. Fiesenheiser (Ill.)	6½
R. H. Sherlock (Mich.)	6½
Chester P. Siess (Ill.)	6½
George C. Alden (Calif.)	5½
A. Amirikian (Md.)	5
Charles G. Peller (Ind.)	5
Luis A. Pietri-Lavie (Venezuela)	5
Andrew Reti (Venezuela)	4½
E. W. Bauman (D. C.)	4
Earl I. Brown (Ga.)	4
Phil M. Ferguson (Texas)	4
Henry L. Kennedy (Mass.)	4
James A. McCarthy (Ind.)	4
Cesar Oliver-Rugeles (Venezuela)	4
Byram W. Steele (D. C.)	4
Ivan M. Viest (Ill.)	4
Castor Segundo Goa (Venezuela)	3½
Samuel Hobbs (Calif.)	3½
Wm. T. F. Hooper, Jr. (Ill.)	3½
Frank H. Jackson (D. C.)	3½
Melvin W. Jackson (Ga.)	3½
S. Moore (Canada)	3½
Clayton M. Crosier (Kans.)	3
Raymond E. Davis (Calif.)	3
Ashby T. Gibbons, Jr. (Ga.)	3
Gerald K. Gillan (Mo.)	3
Emil A. Gramstorff (Mass.)	3
Walter N. Handy (Ill.)	3
M. D. Hicklin (Ore.)	3
J. E. Jellick (Calif.)	3
G. Millsom (Canada)	3
Oscar A. Nunez (Venezuela)	3
James L. Atkinson (Calif.)	2½
F. Thomas Collins (Calif.)	2½
H. J. Gilkey (Ia.)	2½
Orlando W. Irwin (Ill.)	2½
W. H. Jacobs (Ill.)	2½
S. J. Warberg (Mich.)	2½
George D. Youngclaus (Calif.)	2½
Ralph L. Barbehenn (N. J.)	2
Roberto Barillas F. (Guatemala)	2
Sterling Lowe Bugg (Fla.)	2
W. Fisher Cassie (England)	2
J. J. Creskoff (D. C.)	2
Belmon U. Duval (Ohio)	2
Eddy N. Hernandez (La.)	2
J. Sterling Kinney (N. Y.)	2
George E. Large (Ohio)	2
William Lerch (Ill.)	2
F. J. Ochoa U. (Guatemala)	2
Leroy A. Staples (La.)	2
J. Neils Thompson (Texas)	2
J. F. Baxter (N. Y.)	1½
Oscar Benedetti (Venezuela)	1½
Anthony Henry Clark (England)	1½
Aloysius E. Cooke (Conn.)	1½
Charles W. DeGroff (N. Y.)	1½
Harlan H. Edwards (Wash.)	1½
Rodolfo C. Garcia (Philippine Islands)	1½
H. F. Gonnerman (Ill.)	1½
Ernst Gruenwald (N. Y.)	1½
George E. Hatch (Hawaii)	1½
Elmo C. Higginson (Colo.)	1½
Leo M. Legatski (Mich.)	1½
H. J. McGillivray (Fla.)	1½
J. Neil Mustard (Canada)	1½
Henry A. Pfisterer (Conn.)	1½
James E. Schumann (Calif.)	1½
Thomas C. Shedd (Ill.)	1½
Howard Simpson (Mass.)	1½
John H. Thornton (Scotland)	1½
A. G. Timms (D. C.)	1½
G. K. Viall (Wis.)	1½
Jose Antonio Vila (Cuba)	1½
Charles E. Wuerpel (Ill.)	1½
Roderick B. Young (Canada)	1½
J. B. Alexander (Calif.)	1
F. O. Anderegg (N. J.)	1
Edward L. Ashton (Ia.)	1
Hugh Barnes (Calif.)	1

F. Barona (Mexico).....1
Hans H. Bleich (N. Y.).....1
Woodrow L. Burgess (Ore.).....1
E. F. J. Clark (Canada).....1
Roy R. Clark (Ore.).....1
Kenneth W. Clayton (Australia).....1
W. S. Cottingham (Wis.).....1
M. A. Craven (New Zealand).....1
R. A. Crysler (Canada).....1
Walter T. Daniels (D. C.).....1
Harmer E. Davis (Calif.).....1
Raymond E. Davis, Jr. (Calif.).....1
DeWitt Dieterich (Ill.).....1
A. H. Douglas (Canada).....1
H. B. Emerson (Ill.).....1
C. C. Everhart (Ia.).....1
Stanley Gordon Fisher (Canada).....1
George W. Ford (Fla.).....1
Jose M. Franco-Cancio (Puerto Rico).....1
William E. Gabelman (Calif.).....1
Randall L. Gomien (Ohio).....1
Otto Graf (Germany).....1
Nathaniel Grant (Mo.).....1
Gilberto Guardia (Panama).....1
Horace P. Hamlin (N. Y.).....1
Robert J. Hansen (Mass.).....1
Terrell R. Harper (Texas).....1
W. L. Harrell (Texas).....1
Charles E. Hawke (Canada).....1
Harry H. Hawley (Ohio).....1
Philip W. Helsley (Calif.).....1
Bernard C. Herring (England).....1
Wallace E. Jobusch (Ind.).....1
William R. Johnson (Ore.).....1
Edgar R. Kendall (Nebr.).....1
Thomas B. Kennedy (Miss.).....1
Frank Kerekes (Ia.).....1
John C. King (Ohio).....1
K. V. Kini (India).....1
R. A. Kirkpatrick (N. Y.).....1
Blas Lamberti (Venezuela).....1
Demetrio Lausell (Puerto Rico).....1
David V. Lewin (Ohio).....1
Harry E. Lewis (N. Y.).....1
Alvin C. Loewer, Jr. (Pa.).....1
Winthrop E. Luke (Mass.).....1
Charles L. Luther (Ky.).....1
Ian Macallan (New Zealand).....1
Glenway Maxon (Wis.).....1
G. T. McCoy (Calif.).....1
J. H. Murdough (Texas).....1
J. A. Murlin (Texas).....1
James A. Murray (Mass.).....1
William D. Nowlin (Va.).....1
Carlos Ospina (Colombia).....1
Wm. Otcovsky (Ill.).....1
Kenneth K. Paget (Canada).....1
Jose Antonio Pardo B. (Mexico).....1
Dean Peabody, Jr. (Mass.).....1
Howard F. Peckworth (Ill.).....1
Stanley A. Phillip (Ill.).....1
Harry W. Piper (Md.).....1
Dan H. Pletta (Va.).....1
Herman G. Protze (Mass.).....1
Frederick A. Reickert (Ill.).....1
Theodore O. Reyhner (Colo.).....1
Paul F. Rice (Mich.).....1
Evan L. Richard (Australia).....1
Frank E. Richart (Ill.).....1
Horace B. Rickey, Jr. (La.).....1
Bernard L. Robinson (N. Y.).....1
Gilbert H. Rowe (S. C.).....1
Quentin Rust (Calif.).....1
Otto Sauermann (Ill.).....1
Emil Schmid (N. J.).....1
Wilfrid Schnarr (Canada).....1
Elwyn E. Seelye (N. Y.).....1
E. R. Smallhorn (Canada).....1
Howard R. Staley (Mass.).....1
Kenneth G. Stevens (S. Rhodesia).....1
M. Eugene Sundt (N. M.).....1
Pascal L. Tasio (N. Y.).....1
Wm. H. Thoman (Colo.).....1
H. F. Thomson (Ill.).....1
E. W. Thorson (Colo.).....1
LeRoy A. Thorssen (Canada).....1
G. E. Troxell (Calif.).....1
F. E. Votaw (Mass.).....1
A. G. Watt (Pa.).....1
W. R. Waugh (D. C.).....1
C. A. Willson (N. Y.).....1
Merle D. Wilson (Ohio).....1
S. H. Woodard (N. Y.).....1
A. E. Wynn (S. Africa).....1
Aubrey D. Young (La.).....1
Edward F. Young (Minn.).....1
R. A. Zern (Pa.).....1

The following credits are, in each instance "50-50" with another member.

James Adam, Jr. — Grayson Gill
Herbert C. Allen — A. T. Goldbeck
Frederick G. Anderson — O. H. Gosswein
Charles D. Babcock — Alex H. Graves
Malcolm F. Baker — Homer M. Hadley
C. Merrill Barber — Albert Haertlein
Joseph L. Bargar — Theodore F. Hagerman
George E. Barnes — William H. Hall
Frank Baron — W. C. Hansen
A. Allan Bates — Harvey L. Harris
K. P. Billner — Roy Hawkins
G. L. Blanchard — John M. Hayes
A. E. Bond — Eivind Hognestad
Harry E. Born — W. A. Hohlweg
J. G. Bragg — William N. Holway
James E. Branch — Wilfred M. Honour
Carlos D. Bullock — E. S. Horne
S. D. Burks — J. W. Hubler
R. A. Burmeister — Virgil L. Hughes
Harold J. Buttery — Maurice Hunt
D. J. Cameron — Anthony Hunter
Julian B. Carson — S. E. Hyde, Jr.
Harold Carter — Raja A. Iliya
S. J. Chamberlin — W. R. Ingram
Jules P. Channing — C. E. Ireland
Ralph E. Coblentz — C. L. N. Iyengar
Miles K. Cooper — Arthur M. James
Roger H. Corbetta — M. E. James
William A. Crabb — Bruce M. Johnson
A. E. Cummings — J. G. Johnson
Clayton L. Davis — H. D. Jolley
G. A. DeLong — Bernerd D. Keatts
Harry Delzell — J. W. Kelly
R. G. Douglas, Jr. — Clyde Kesler
Albert W. Dudley — W. D. Kimmel
Leonard E. Dunlap — W. P. Kinneman
John R. Dwyer — Ralph W. Kluge
Harry Ellsberg — Theodore C. Knight
W. J. Emmons — Jean H. Knox
L. G. Farrant — M. C. Kolinski
Jorge Figuls — Paul P. Kraai
John D. Freeman — Richard J. Kroc
P. J. Freeman — W. G. Limbach
Bernardo Fuentes A. — J. E. Lothers
Mario W. Gamalero — Irvin H. Luke
J. H. Gandhi — I. D. MacKenzie
A. H. Garnsey — M. F. Macnaughton

Continued on p. 26

ACI Medalists

Continued from p. 4

H. F. Thomson—Wason Medalist

H. F. Thomson, Manager, Redi-Mix Concrete Division, Material Service Corp., Chicago, Ill., has been prominently connected with the ready-mixed concrete industry for many years.

He has been active in ACI affairs since 1928 and with a one year lapse—1941—has been a member of the ACI Board of Direction since 1939. He was elected vice-president of the Institute in 1949 and was nominated for president last fall for the term beginning in February 1951, to succeed Frank H. Jackson.

After graduating from Washington University in 1910, Mr. Thomson served as instructor and research worker at the Massachusetts Institute of Technology for four years. He was later associated with the Provident Chemical Works, successively as assistant superintendent, Eastern manager in New York City, sales manager, and (from 1920-24) vice-president. In 1924 he joined the Federal Phosphorous Co. as executive assistant.

In 1927 Mr. Thomson became vice-president of the General Material Corp. of St. Louis, one of the pioneer companies in producing commercial ready-mixed concrete. He continued with that company as vice-president and subsequently as president until 1948 when he was retained by Material Service Corp. to be in over-all charge of their extensive ready-mixed concrete service in the Chicago area.

Mr. Thomson has served as president of the National Ready Mixed Concrete Assn. and is a former national director of ASCE.

PROFESSIONAL CARD

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Membership certificates

Repeated but infrequent requests for membership certificates indicate that enough members may want them to warrant having them made. At its fall meeting, the Board of Direction authorized the Secretary to determine through this notice whether or not the Institute would be justified in going to the expense of having the necessary work done.

It is estimated that certificates suitable for framing can be furnished ACI Members at a cost of not over $3.50. Those interested are requested to clip the attached coupon and mail it to Institute headquarters. *Please do not send money until billed.* If there are not enough interested, the Institute would not be justified in supplying certificates to a few at a loss.

American Concrete Institute
18263 W. McNichols Road
Detroit 19, Mich.

Gentlemen: Please place an order for one membership certificate to be furnished provided enough members buy them to make it worthwhile at a cost not to exceed $3.50.

Signed.....

Building Research Congress program announced

The Building Research Congress, to be held in London, England, Sept. 11-20, 1951, will be divided into three divisions: the engineering and structural aspects of building; the second division concerned with building materials and the third division to deal with the factors affecting the comfort and efficiency of the people using the buildings.

Of special interest to ACI Members will be papers on reinforced concrete developments, prestressed concrete developments, design of thin concrete shells, concrete quality control, lightweight concrete, curing methods and durability of concrete structures.

The conference is sponsored by the British professional institutions, technical societies and governmental departments, with the support of representative industrial federations in Great Britain.

PCA elects new officers

Smith W. Storey, president of the General Portland Cement Co. and the Consolidated Cement Corp. of Chicago, Ill., was elected chairman of the Board of Directors of the Portland Cement Assn. at its annual meeting in Chicago. Mr. Storey succeeds Walter C. Russell, president of Peerless Cement Corp., Detroit, Mich.

Four new directors were also elected. They include Donald S. MacBride, president, Hercules Cement Corp., Philadelphia, Pa.; F. A. Weibel, president, Allentown Portland Cement Co. and vice-president, Valley Forge Cement Co., Catasauqua, Pa.; Eugene D. Hill, president, Louisville Cement Co., Louisville, Ky.; and Harold M. Scott, president, Keystone Portland Cement Co., Philadelphia, Pa.

Honor Roll

Continued from p. 24

James V. Mandia
Eastman M. Markell
W. T. McClenahan
Sam McCluer
C. T. J. McDowell
A. J. McElrath
Douglas McHenry
Harmon S. Meissner
Willis T. Moran
E. J. Mueller
Glenn Murphy
V. S. Murray
A. E. Myers
William T. Neelands
Henry L. Neve
Walter Newland
Roy A. Nyquist
Raymond G. Osborne
John P. Ottesen
Douglas E. Parsons
Orley O. Phillips
Thomas Poli
Milos Polivka
James A. Polychrone
Chesley J. Posey
Leo F. Pratt
Walter H. Price
Charles J. Prokop
Raymond C. Reese
Ross M. Riegel
Guy Rinfret
Charles S. Rippon
Richard A. Roberts
A. Rodriguez-Delfino
Pedro Rodriguez
Paul Rogers
A. W. Rohlwing
Mark R. Rosumny
Oswaldo Rovati B.
William H. Rowan
John A. Ruhling
Frederic Rusche
Nicholas R. Samaha
Robert L. Sanks
R. W. Sauer
D. M. Schmid
Axel Schulze
James M. Shivley
J. Morgan Smith
Myron M. Smith
Ernest L. Spencer
R. A. Spencer
R. W. Spencer
John J. Stedje
John D. Stites
Herbert M. Stoll
Hale Sutherland
Ernest O. Sweetser
Thomas F. Taylor
L. W. Teller
Walter F. Tews
George P. Thigpen
J. Trueman Thompson
B. K. Thornley, Jr.
T. Thorvaldson
Paul M. Trueblood
Lee Turzillo
Lewis H. Tuthill
I. L. Tyler
Miguel Villa
E. H. Walker
Julius J. Warner
G. E. Warren
Willard W. Warzyn
L. T. Willoughby
Clement T. Wiskocil
Francis P. Witmer
Bourdette R. Wood
H. D. Worthington

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Vermiculite specifications

The Vermiculite Institute has revised its specifications for roof decks and roof fill insulation. The new specifications are contained in an 8-page booklet which includes details formerly issued separately.

A new section on ready-mixed concrete has been added and a new tabulation consolidates data on vermiculite concrete. Copies can be obtained from Vermiculite Institute, 208 S. LaSalle Street, Chicago 4, Ill.

Form stakes

Rigid corrugated steel stakes for concrete formwork that can be reused indefinitely have been introduced to the California construction industry. The stakes are formed and pointed for easy driving through gravel or hard formations and predrilled nail holes allow ready attachment of horizontal formers. Literature is available from Walter B. Snook Enterprises, 751 YW Loma Verde Ave., Palo Alto, Calif.

Standards are your business

"Standards Are Your Business," a pamphlet published by the American Standards Assn., explains what ASA is and does. It emphasizes the value of standards of nationwide scope and tells how these national standards mean dollar savings to manufacturer and consumer alike.

The booklet is available from American Standards Assn., 70 East 45th Street, New York 17, N. Y.

Join
MARCH OF DIMES
JANUARY 15-31

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 700-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.

The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.

The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.

A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.

The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.

The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly anchorages; uses large diameter wires (⅛-in. diameter now available on the market), instead of the customary ⅛-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

Proposed Revision of BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-47).....47-18

Price 35 cents.

REPORT OF COMMITTEE 318—Dec. 1949, pp. 269-276 (V. 47)

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special achorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

CURING CONCRETE PAVEMENTS WITH MEMBRANES.....47-19

Price 35 cents.

C. C. RHODES—Dec. 1949, pp. 277-296 (V. 47)

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensive as wet curing under the same conditions.

BLADE CHANGES IMPROVE TILTING MIXER.....47-20

Price 35 cents.

GLENWAY MAXON—Dec. 1949, pp. 297-300 (V. 47)

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

PROTOTYPE PRESTRESSED BEAM JUSTIFIES WALNUT LANE BRIDGE DESIGN.....47-21

Price 35 cents.

GUSTAVE MAGNEL—Dec. 1949, pp. 301-316 (V. 47)

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

FINISHING AND CURING: A KEY TO DURABLE CONCRETE SURFACES.....47-22

Price 35 cents.

MYRON A. SWAYZE—Dec. 1949, pp. 317-332 (V. 47)

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

ECONOMY THROUGH BETTER CONTROL OF REINFORCING STEEL.....47-23

Price 35 cents.

F. TESSITOR and P. ROSEWARNE—Dec. 1949, pp. 333-340 (V. 47)

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel used, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

Proposed Revision of MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI 315-48).....47-24

Price 35 cents.

REPORT OF COMMITTEE 315—Jan. 1951, pp. 349-352 (V. 47)

Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

COARSE-GROUND CEMENT MAKES MORE DURABLE CONCRETE.....47-25

Price 35 cents.

HAROLD W. BREWER and RICHARD W. BURROWS —Jan. 1951, pp. 353-360 (V. 47)

A test procedure for mortar ring specimens is described. Rings containing coarse-ground cement shrunk less and showed greater resistance to freezing and thawing and to outdoor exposure than those containing fine-ground cement. These laboratory tests indicate that coarse-ground cement produces more durable concrete than fine-ground cement.

SIMPLE EQUIPMENT ECONOMICALLY EXPLORES PRESTRESSING..47-26

Price 35 cents.

MARVIN L. MASS and JACK R. JANNEY—Jan. 1951, pp. 361-364 (V. 47)

A tubular grip utilizing type metal to hold the prestressing wires is described. Other equipment includes a framework against which the prestressing force is applied and adjustable formwork for experimental beams. The equipment is so simple that it can be easily and inexpensively built to permit study of prestressed reinforced concrete in the small laboratory.

LABORATORY TESTS OF SPACED AND TIED REINFORCING BARS....47-27

Price 35 cents.

WILLIAM T. WALKER—Jan. 1951, pp. 365-372 (V. 47)

Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little

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or no advantage would be obtained by spacing deformed reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which interlocking of lugs could take place there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Twelve beams containing three different types of deformed bars either spaced or tied and extending the entire length of the specimens were tested. The sixty vertical pull-outs tested also contained three different types of deformed bars. Half of these were made with the bars spaced and half with the bars tied together. The reinforcing arrangement brought only one bar out of the specimens for pulling; the other was allowed to bear on the base plate.

Spacing of the bars in both beams and pull-outs conformed to the joint code minimum spacing requirement of 2½ diameters center to center with a minimum clear distance of 1½ times the maximum size of coarse aggregate.

FINISHING AIR-ENTRAINING CONCRETE PAVEMENTS 47-28

Price 35 cents.

CHARLES W. ALLEN—Jan. 1951, pp. 373-376 (V. 47)

Based on a survey of state highway department practices, common difficulties in finishing air-entraining concrete pavements are enumerated. Practices in adjusting transverse finishing machines are given and revised finishing practices are discussed. Delayed finishing, as specified in several states, is considered in connection with the riding qualities of the finished surfaces. On the basis of experience in Ohio it is concluded that smooth pavements can be built of air-entraining concrete without delayed finishing.

EFFECTS OF LATERAL LOADS ON ARCHES 47-29

Price 35 cents.

JAMES P. MICHALOS—Jan. 1951, pp. 377-388 (V. 47)

The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shears in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced with struts normal to the ribs. Numerical studies are included.

GETTING MORE FOR OUR CONCRETE DOLLAR 47-30

Price 35 cents.

I. E. MORRIS—Jan. 1951, pp. 389-396 (V. 47)

A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. The system is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. The members are designed to use both concrete and steel to maximum advantage.

DISCUSSION

Discussion closed January 1, 1951

Sept. Jl. '50

Some Observations on the Use of Reinforcing Steel in Concrete Pavements—Bengt F. Friberg

Tests of Paper Molds for Concrete Cylinders—Robert A. Burmeister

Admixtures in Concrete—W. T. Moran, F. H. Jackson, Bruce E. Foster and T. C. Powers

Improved Sonic Apparatus for Determining the Dynamic Modulus of Concrete Specimens—C. E. Goodell

Analysis of Three-Dimensional Beam-and-Girder Framing—Phil M. Ferguson

A general summary of conditions affecting concrete strength.

Factors Influencing Concrete Strength

By WALTER H. PRICE†

SYNOPSIS

The effect of mix proportions, type and brand of cement, availability of moisture for curing, accelerators and curing temperatures on the rate and potential strength development of concrete are discussed. The influence of rate and frequency of load applications, dimensions of test specimens and lateral restraint on the indicated strength are also discussed, and information is furnished on the variations in strength which might be expected on a typical job. Compressive, tensile, flexural, bond and shearing strengths are compared, and the strengths of control cylinders are compared with the strengths of cores drilled from structures at later ages. Information is also furnished on strength loss from freezing and thawing and alkali-aggregate expansion.

INTRODUCTION

The working stresses used in concrete design, with the possible exception of pavements, are usually based on the 28-day compressive strengths of 6 x 12-in. control cylinders made from representative samples of the concrete and moist cured at 70 F for the full 28-day period. This practice has led to a premium being placed on the development of high strengths at the early age of 28 days. Cement manufacturers point their product toward this end, and accelerators are sometimes employed to increase the 28-day strengths of the concrete. The production of high 28-day strengths through the use of accelerators, high curing temperatures, or high early strength cement usually results in comparatively lower strengths at later ages. Furthermore, concrete of comparatively low strength containing entrained air may be much more resistant to weathering than a stronger concrete containing no entrained air, and strength alone does not indicate how well a structure will resist the elements.

It is well-known that the strength of the concrete in the structure may be considerably different from that indicated by the 28-day control cylinders

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†Member American Concrete Institute, Head, Materials Laboratory Section, Research and Geology Division, U. S. Bureau of Reclamation, Denver, Colo.

due to different curing conditions. Also, even if the structure and control cylinders were cured under identical conditions the control cylinders would not be completely indicative of the strength of the concrete in the structure because of differences in size, rate of loading, and lateral restraint.

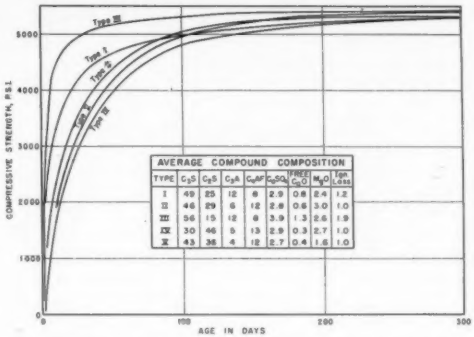
Usually where moisture is available for curing or where moisture contained in the concrete is not lost through drying, the strength development of the concrete will continue for a number of years. This later strength development adds to the safety of the structure but may be of little value where the structure has to support the full load at an early age. Consideration is given to the later strength development of concrete in the selection of mix proportions for dams which take a long time to construct and where the reservoir is not filled until possibly a year after placing the concrete. In the case of reinforced concrete buildings which are usually not moist cured and where the members receive no moisture after construction, the concrete in the structure may never reach the strength indicated by the 28-day moist cured specimens. Designers should consider this in choosing working stresses.

MATERIALS AND MIX PROPORTIONS

The strength of concrete is determined to a large extent by the quality and proportions of materials used in its fabrication. The quality and particle shape of the aggregate have a marked effect on the strength of the concrete, but unless the aggregate is very weak, such as that used in the production of lightweight concrete, it is usually possible to obtain a desired strength by increasing the cement content. Cement is the most important ingredient of the mix, because it binds the aggregate particles together and because the proportions of water and cement used in the fabrication of concrete have such a marked effect on the strength and other properties of concrete.

There are five major types of portland cement covered by ASTM and Federal specifications. Type I is the one normally used in concrete construction, the other types are produced to meet certain special requirements; Type II for moderate heat evolution during hydration, Type III for high early strength, Type IV for low heat evolution, and Type V for high resistance to sulfate attack. Cements of comparatively high lime content (high C_3S) develop high early strength. Finely ground cements also develop strength faster than coarser ground cements. Type III cement is made by increasing the lime content of Type I clinker and by grinding finer. The lime content is generally lower and the silica content higher in the other special cements and the amount of calcium aluminate (C_3A) is held below a specified maximum for these cements. The average compound compositions of the five types of cement are shown in the table on Fig. 1. Fig. 1 also shows typical strength development of concretes made and cured under similar conditions for the five types of cement. Large differences in strength are obtained at early ages for concretes made with the different types of cement, but the strengths tend to approach the same value at later ages.

In recent years attention has been directed to portland-pozzolan cements, which are mixtures of portland cement and certain chemically active natural or artificial materials called pozzolans. The rate of strength development of concrete made with pozzolans is dependent on the activity of the pozzolan and the proportions used in the mix. Most portland-pozzolan cements compare with low-heat portland cements in that they harden slowly and require a longer period for curing.



AVERAGE COMPOUND COMPOSITION

TYPE	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	FREE CaO	MgO	Ign. Loss
I	49	25	12	8	2.9	0.8	2.4	1.2
II	46	29	6	12	2.8	0.6	3.0	1.0
III	56	15	12	8	3.9	1.3	2.6	1.9
IV	30	46	5	13	2.9	0.3	2.7	1.0
V	43	36	4	12	2.7	0.4	1.6	1.0

Fig. 1—Strength gain of concrete containing different types of cement

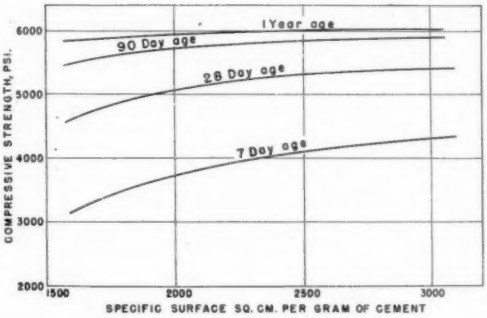


Fig. 2—Strength of concrete at earlier ages is greater for finer ground cement

The effect of cement fineness on the strength development of concrete is shown in Fig. 2. The fineness is expressed as square centimeters of particle area per gram of cement, and the larger the area the finer the cement. Fine cements are undesirable because they contribute to cracking and low resistance to freezing and thawing. American cements are ground to a fineness of about 1800 sq cm per g (Wagner) because the cement is sold on the basis of 28-day strength development which designers and contractors have become accustomed to using as a yardstick of concrete quality. Also, higher early strength permits earlier stripping of forms and faster construction progress. Finer cements have the advantage of producing more workable concrete. However, from the durability standpoint, coarser cement is more desirable.

Although the strength of concrete made with one type of cement can usually be expected to fall within a certain range, as is shown in Fig. 1, there may be greater differences between cements of the same type from different mills than between types, as shown in Fig. 3. Fig. 3 also shows that in certain cases Type IV cement, which is ordinarily a slow strength-developing cement, may produce higher strengths at 28 days age than a Type I cement.

It is generally known that the strength of concrete is decreased by increas-

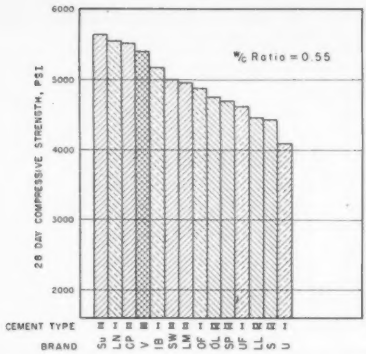


Fig. 3—Variation in strength of comparable concrete mixes made with 14 different brands of cement

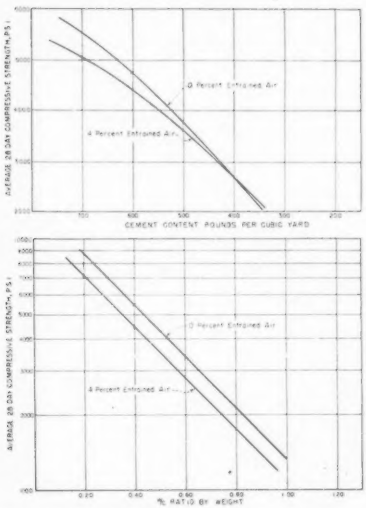

0 Percent Entrained Air
4 Percent Entrained Air
CEMENT CONTENT, POUNDS PER CUBIC YARD
0 Percent Entrained Air
4 Percent Entrained Air
W/C RATIO BY WEIGHT

Fig. 4—Effect of air content on compressive strength of concrete

ing the slump of the mix when no additional cement is added, because the paste is diluted. The quality of paste, as measured by the ratio of the water and cement used in the mix, is a good index of potential strength development of the concrete. In the bottom of Fig. 4 the relationship between water-cement ratio and strength is shown.

During recent years it has been found that through the purposeful entrainment of very small dispersed air bubbles in the concrete by adding an agent at the mixer or to the cement, the workability of the fresh concrete and the durability of the hardened concrete are greatly increased. The entrainment of air in concrete, however, usually decreases the strength of the concrete. Where the cement content of the mix is held constant the decrease is not so great and the strength may be actually increased by the entrained air in the range of leaner mixes, as shown in the top portion of Fig. 4. The bottom portion of Fig. 4 compares the strengths of air-entraining concrete and non-air-entraining concrete for concretes having the same water-cement ratio. Generally, the strength of the concrete is reduced about 5 percent for each percentage of air entrained when the water-cement ratio is held constant. It is usually desirable to maintain the air content of concrete below 6 percent because higher percentages do not add materially to the durability, and the drying shrinkage is usually increased if the air content exceeds this figure.

The maximum size aggregate used in the mix has an important effect on the compressive strength produced for a given amount of cement. Where the cement content per cu yd of concrete is maintained constant the compressive strength of the concrete is increased as the maximum size of the aggregate is increased. Where the water-cement ratio is held constant and

a given strength is desired, the cement content is reduced as the maximum size of aggregate used in the mix is increased. About two more sacks of cement per cu yd of concrete are required to produce a given strength for concrete containing aggregate graded up to ¾-in. maximum than for concrete containing aggregate graded up to 6-in. maximum size. For approximately the same compressive strength the difference in cement content for concrete containing aggregate graded up to ¾-in. maximum and one containing aggregate graded up to 1½-in. maximum size is about ⅔ sacks per cu yd of concrete. Some tests have shown that the flexural strength of the concrete is not increased with larger maximum sized aggregate even though the water-cement ratio is reduced through its use.

CURING

Temperature and moisture have pronounced effects on the strength development of concrete. Fig. 5 shows that the development of strength stops at an early age when the concrete specimen is exposed to dry air with no previous moist curing. Concrete exposed to dry air from the time it is placed is about 42 percent as strong at 6 months as concrete continuously moist cured. Specimens cured in water at 70 F were found to be stronger at 28 days than those cured in a fog room at 100 percent relative humidity. The richer mixes showed up to better advantage than the leaner ones under water curing. The strength of the water-cured specimens was about 10 percent higher than the fog-cured specimens for concretes having water-cement ratios of 0.55 by weight. Fig. 5 should be of particular interest to designers of bridges and buildings which are not exposed to moisture and which are not cured during construction. Concrete in many buildings is not cured because of objections by the workmen to water dripping over the work and because

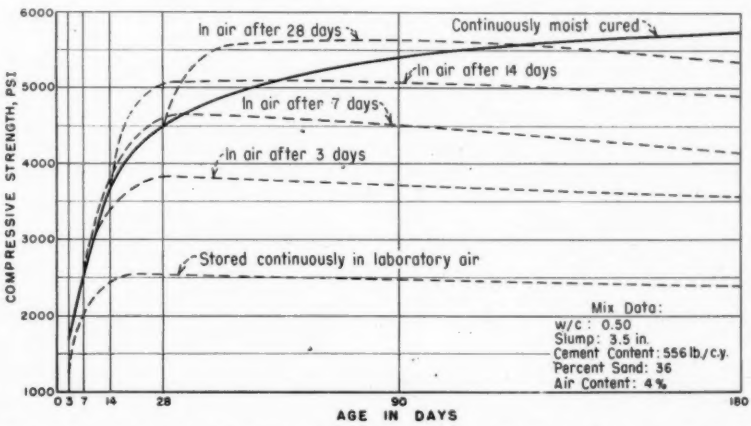


Fig. 5—Compressive strength of concrete dried in laboratory air after preliminary moist curing

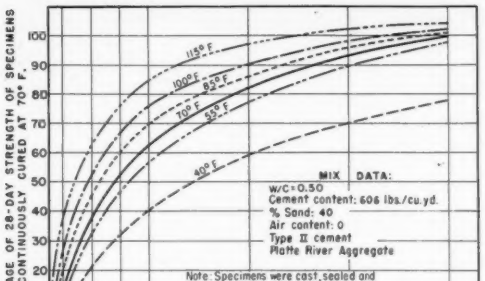


Fig. 6—Effect of curing temperature on the compressive strength of concrete

curing compounds cannot be used due to the necessity of bonding the following lift or floor topping to the concrete already placed.

Curing temperatures have a marked effect on the strength development of concrete, as is shown by the test results presented in Fig. 6 and 7. The results shown in Fig. 6 were obtained by casting and curing the concrete at the temperatures listed on the curves, and under such treatment the highest temperatures developed the highest 28-day strength. For the curves shown in Fig. 7 the concrete was cured at 70 F after holding the specimens at the casting temperature shown for two hours. Such treatment produced opposite results from those shown in Fig. 6, as the specimens made at the lowest temperature produced the highest 28-day strength. The results shown in Fig. 7 agree with those obtained on some Bureau of Reclamation projects where the strength of the field control cylinders was found to be lower during the hot summer months than during the cooler months, even though they were moist cured at about 70 F in every case. Apparently the concrete is weakened by very rapid setting which is not overcome by the subsequent curing at 70 F. Continued curing at higher temperatures for the full 28-day period, as was done for some of the sets of specimens shown in

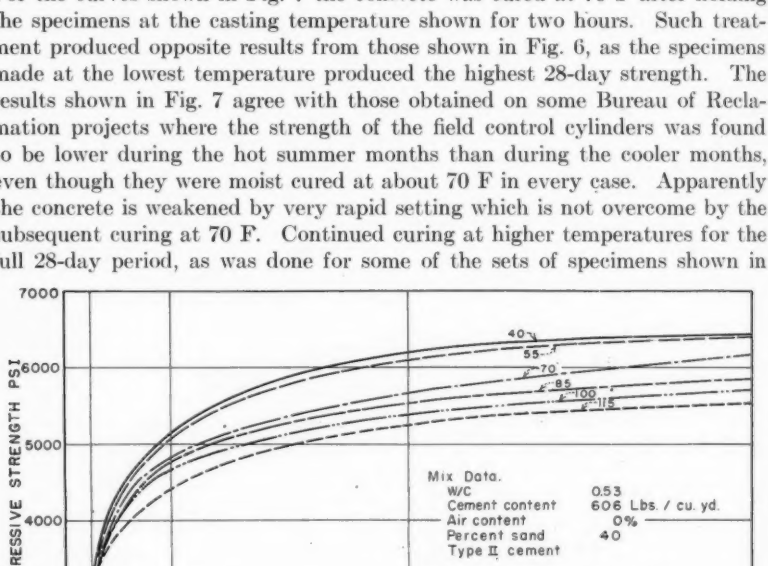


Fig. 7—Effect of initial temperature on the compressive strength of concrete

Fig. 6, accelerated the strength development sufficiently to produce the highest strength for the highest temperature. At later ages, however, the specimens made and cured at higher temperatures had lower strengths than those made and cured at lower temperatures.

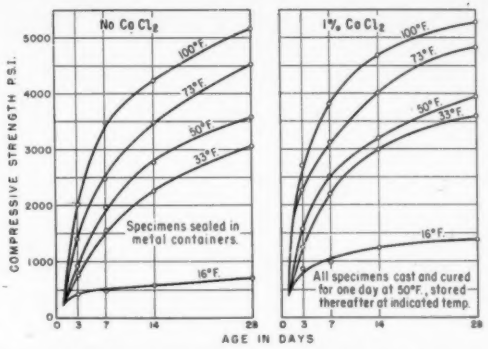


Fig. 8—Compressive strength of concrete cured 24 hours at 50 F and stored at various temperatures. Concrete contained Type II cement and 4 percent entrained air

It is difficult to determine the strength gain of concrete cured at temperatures below freezing because time is required to freeze the specimen at the start and thaw it before breaking. However, some gain in strength at temperatures as low as 16 F is indicated when the concrete was cured at a temperature above freezing for at least one day, as shown in Fig. 8. At temperatures just above freezing the gain in strength is quite rapid. For concrete made and maintained at 50 F for one day and then stored at a temperature of 33 F for the remainder of the 28-day period the strength gain approached that of the concrete cured at 73 F. One percent calcium chloride had a material effect in increasing the strength of the concrete cured at 16 F. This series of tests, which was made in the laboratories of the Bureau of Reclamation, showed that freezing an air-entraining concrete after three days curing above freezing did no damage and that the concrete gained strength rapidly under moist curing conditions when the temperature again rose above freezing.

ACCELERATORS

Calcium chloride is the most common accelerator used to hasten the strength development of concrete. It is commonly used during cold weather to accelerate the set so that the forms can be removed sooner and to make the concrete more resistant to freezing temperatures during its early age.

The results of recent tests made with various percentages of calcium chloride at 40 F and 70 F are shown in Fig. 9. It can be concluded from these results that at temperatures around 40 F the strength of the concrete is improved at all ages by the addition of calcium chloride and that this improvement increases as the percentage of calcium chloride is increased up to 3 percent by weight of the cement which was the maximum amount used in this test series. It is indicated, however, that there is little advantage in using more than 3 percent even at this low temperature, as there was very little increase in strength realized by increasing the percentage of calcium chloride from 2 to 3 percent. For those concretes made and cured at 70 F the very early

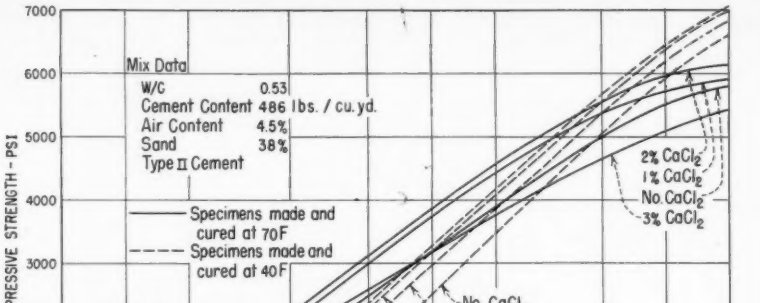


Fig. 9—Effect of small additions of calcium chloride on the compressive strength of concrete

strength development of the concrete was increased as the percentage of calcium chloride was increased up to 3 percent. The 28-day strength of the concrete made at 70 F and containing 3 percent calcium chloride was about 2 percent lower than that of the concrete containing no calcium chloride and about 12 percent lower than the concrete containing 2 percent calcium chloride, indicating that as the placing temperature increases the percentage of calcium chloride must be decreased for optimum results. Less than ½ percent calcium chloride usually retards set. It reacts differently with different cements, and with some cements up to 1 percent may act as a retarder.

Steam is commonly used for accelerating the strength development of precast units such as irrigation pipe, piles, and building blocks. Fig. 10 shows the effect of steam curing on strength of concrete. These data were obtained on 6 x 12-in. concrete cylinders cured through a temperature cycle similar to that used by manufacturers of precast units. This cycle consisted of a 3-hour delay prior to start of steam curing at which time the temperature of the concrete was raised at about 40 F per hour to the desired temperature. At the end of 3 days of steam curing at the desired temperature the specimens were slowly cooled before removing them from the steam room. Molds were removed at the end of the steaming period. These curves show that early strengths increase with the temperature but concrete cured at 200 F showed only slightly higher strengths at early ages than concrete cured at 165 F, and the concrete cured at 200 F gained but little strength after removal from the steam room. Other tests in this series indicate that the concrete can be severely damaged by steaming at high temperature immediately after casting and by rapid temperature rise. This agrees with the data in Fig. 7 which

show that the concrete is weakened by high placing temperatures and rapid setting.

NORMAL FIELD VARIATIONS

Fig. 11 shows the variation in strengths of field control cylinders made and tested on a typical Bureau of Reclamation project. No matter how well a job is controlled there will be some variation in the strength, and the number of values above and below the average will fall in some pattern similar to that shown in Fig. 11. Where there is good control the values are bunched close to the average and the curve is steep, but where there is poor control the values are spread out laterally and the curve is flatter. The distribution of the points tends to follow a normal probability curve which can be used conveniently for measuring the degree of control being obtained on a job and for predicting what average must be obtained to have all strengths fall above a selected figure. The radius of gyration of the points under the probability curve about its center is called the standard deviation. The standard deviation is a measure of the spread of the results. The standard deviation divided by the average strength is called the coefficient of variation. A high coefficient indicates poor control and a low one indicates good control. A coefficient of about 10 percent is about as low as can be expected for field concrete work, and coefficients of as high as 25 percent are not uncommon. For the data shown in Fig. 11 the coefficient of variation is 12.6 percent. This means that ⅔ of all the strengths fall within 12.6 percent of the average, and from the probability curve it is very likely that one out of every 400 specimens will be 3 × 12.6 percent removed from the average.

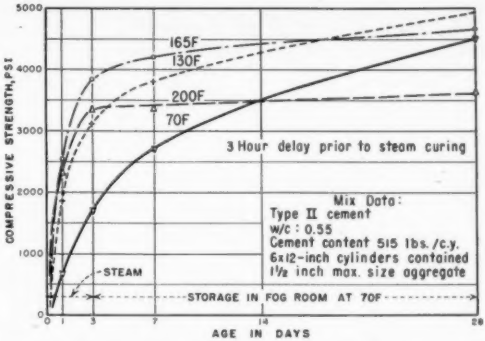


Fig. 10—Compressive strength of specimens steam cured 3 days at various temperatures followed by moist curing at 70 F

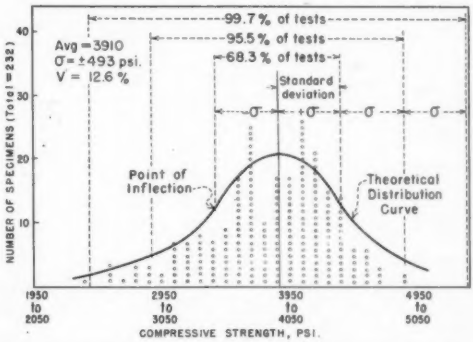


Fig. 11—Typical frequency distribution of strength of 6 x 12-in. control cylinders (Angostura Dam—1948 season)

Table 1—TYPICAL STRENGTH, CEMENT, AND COST REQUIREMENTS FOR DIFFERENT CONTROL STANDARDS. Control strength 2500 psi.

Percent of tests higher than control strength	Coefficient of variation, percent	Average strength required, psi	Est. cement content, lb per cu yd (1½-in. max. agg.)	Estimated cost of cement per cu yd of concrete*
70	5	2580	392	$5.21
	10	2650	395	5.26
	15	2710	404	5.37
	20	2800	413	5.50
	25	2880	424	5.64
80	5	2610	394	5.24
	10	2750	407	5.41
	15	2850	420	5.59
	20	3010	440	5.85
	25	3180	455	6.05
90	5	2660	400	5.32
	10	2910	428	5.69
	15	3090	448	5.96
	20	3380	474	6.30
	25	3720	526	6.99
99	5	2830	417	5.55
	10	3310	476	6.33
	15	3840	540	7.18
	20	4770	655	8.71
	25	6250	1000	13.30

*Based on 1,000,000 cu yd quantities of concrete and cement at $5.00 per barrel

In Table 1 statistical methods have been used to show what average strengths are required to have 70, 80, 90, and 99 percent of the strength values fall above a selected value such as 2500 psi. It should be noted that under good control, with a coefficient of variation of 10 percent, an average strength of only 2910 psi is required to have 90 percent of the strengths fall above 2500 psi, but that an average strength of 3720 psi is required for a coefficient of variation of 25 percent. This demonstrates the economy of good control.

EFFECT OF HEIGHT AND DIAMETER OF TEST SPECIMENS, TYPE OF CAP AND TYPE OF MOLD ON INDICATED COMPRESSIVE STRENGTH

The indicated strength of concrete made and cured under the same conditions is different when tested in cylinders of different height and diameter. Fig. 12 shows the effect of diameter of cylinder on the indicated strength of the concrete. This shows the strength of a 36-in. diameter cylinder as being only 82 percent that of a 6-in. diameter cylinder. This difference is believed due to the possible faster strength gain of the smaller diameter cylinders. The strengths of 10-in. diameter cores and 22-in. diameter cores drilled from five year old concrete of Shasta and Friant Dams test blocks were practically the same, indicating that age may equalize difference due to difference in diameter of specimen. The data indicate that large masses of concrete in a structure cannot be expected to approach the 28-day strength of

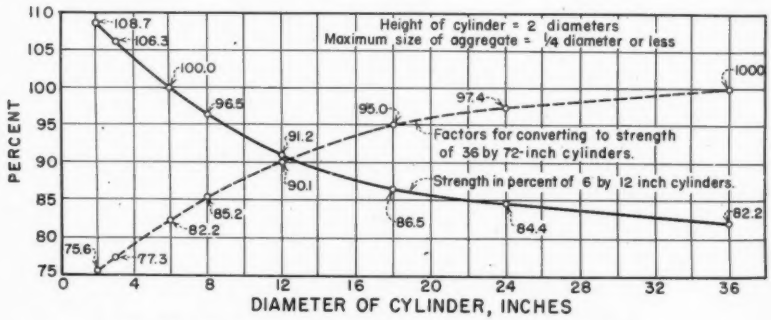


Fig. 12—Effect of cylinder size on compressive strength of concrete

concrete as indicated by a 6 x 12-in. cylinder for a number of months. Fig. 13 shows the well-known effect of height of specimen on the indicated strength.

The indicated strength of concrete capped with soft or rubbery material can be as much as 50 percent lower than companion cylinders capped with a hard, strong material. Cylinders capped with neat portland cement, Lumnite cement or a mixture of three parts of sulfur and one part of fire clay produced strengths equal to those obtained on cylinders with the ends ground to within 0.002 in. of a plain surface. The strengths of cylinders capped with plaster of paris were about 5 percent lower than those with ground ends. Where the testing machine is equipped with a movable spherical head the top of the cylinder surface may slope as much as 3° from the horizontal without affecting the indicated strength. Large aggregate projecting above the top surface of the cylinder seems to cause lower strength even when the surface is leveled by capping.

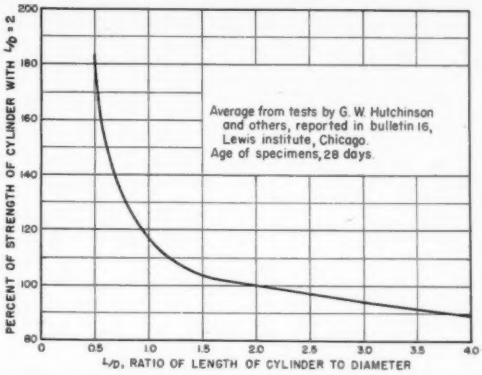


Fig. 13—Relation of length and diameter of specimen to compressive strength

Specimens made in cardboard molds usually develop lower strengths than those made in steel molds. In recent tests made in the Denver laboratories of the Bureau of Reclamation the compressive strength of specimens formed in cardboard molds was found to be about 3½ percent less than companion specimens formed in steel molds. A total of 260 cylinders were cast in this study, 130 in steel molds and 130 in cardboard molds. Only one type of mold having a cardboard bottom and side was tested; the specimens were placed in the fog room immediately after casting, where they remained until tested for strength. The molds were removed from the specimens 24 hours after casting. Burmeister,[11] showed the strengths of specimens cast in cardboard molds to average about 9.5 percent less than those cast in steel molds.

EFFECT OF RATE AND FREQUENCY OF LOAD APPLICATION AND LATERAL RESTRAINT ON THE INDICATED COMPRESSIVE STRENGTH OF CONCRETE

The rate at which a concrete specimen is loaded has a pronounced effect on the indicated compressive strength. Fig. 14 shows that if a specimen is loaded slowly it will break at a much lower load than when it is loaded at the standard rate and that concrete can be expected to withstand indefinitely only 70 percent of its ultimate load, as determined by the standard rate of loading.

Concrete will sustain an indefinite number of repeated applications of stress where the maximum stress is maintained at a value of 50 percent below the

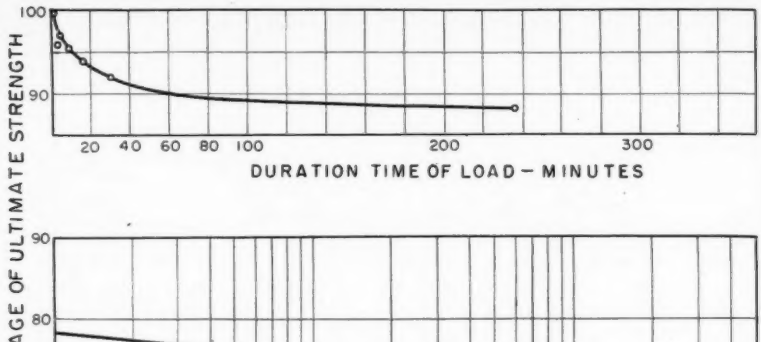


Fig. 14—Top—Concrete will support ultimate load as determined by standard procedure for only a short period. Bottom—Extrapolation of top curve. Concrete will support only about 70 percent of ultimate load for an indefinite period

ultimate strength of the concrete. The 50 percent figure was found to hold for both flexural and compressive stresses. The number of repetitions of loading the concrete will sustain decreases rapidly as the stress is increased above 50 percent of the ultimate strength of the concrete and for a stress of 70 percent of the ultimate, concrete will sustain only about 5000 repetitions before failure.

Concrete when restrained on all sides will support a much higher load than unrestrained concrete. Fig. 15 shows that the axial load specimens will support increases as the lateral restraint is increased. An extrapolation of the curve to the right of the ordinate indicates that a small amount of tension materially reduces compressive resistance at right angles to the tensile force.

RELATIONSHIPS OF COMPRESSIVE, TENSILE, FLEXURAL, BOND AND SHEARING STRENGTHS

Table 2 shows the relationship between the compressive strength, tensile strength, and flexural strength of concrete. The ratio of tensile to compressive strength decreases as the compressive strength increases and approaches a constant of about 7 percent for higher compressive strengths. Other published data indicate that the ratio of tensile to compressive strength decreases as the age of the concrete increases to the same constant value of 7 percent.

The indicated bond strength of concrete to steel reinforcement is influenced materially by test procedures, type of reinforcement, and properties of the

concrete. Fig. 16 shows in a qualitative way the relationship between bond strength and compressive strength of the concrete for plain and deformed bars. The ratio of bond strength to compressive strength decreases as compressive strength increases. For the curves shown the bond strength of the deformed bars is 24 percent of the compressive strength for 2000 psi concrete and 18 percent of the compressive strength for 5000 psi concrete. The corresponding values for the plain bars are 13 and 10 percent, respectively. The relationship of bond to compressive strength apparently is not changed materially by air entrained in the proportions recommended.

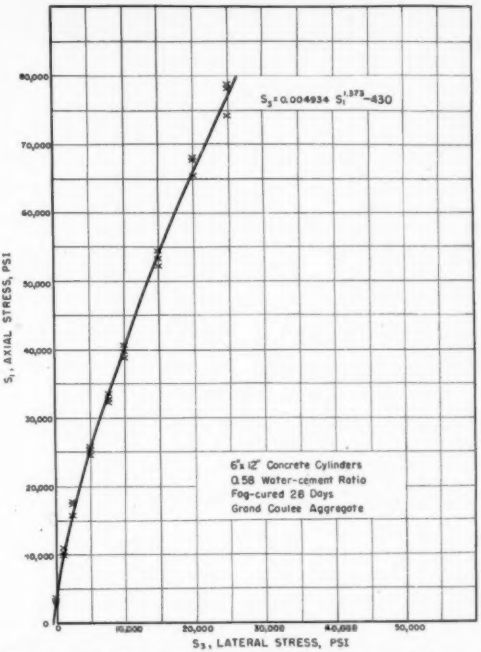


Fig. 15—Relation of axial stress to lateral stress at failure in triaxial compression tests of concrete

The shearing resistance of concrete is usually considered as the sum of the cohesive strength plus the internal frictional resistance of the concrete along the plane of slip. The cohesive strength of concrete is reported as varying from 20 to 40 percent of the compressive strength. The values in the lower range are indicated as being more reliable and the average is probably about 25 percent of the compressive strength. Shearing resistance and cohesive strength are equal where there is no stress normal to the plane of slip.

TABLE 2—COMPARISON OF COMPRESSIVE, FLEXURAL AND TENSILE STRENGTH OF PLAIN CONCRETE*

Strength of plain concrete, psi			Ratio, percent		
Compressive	Modulus of rupture	Tensile	Modulus of rupture to compressive strength	Tensile strength to compressive strength	Tensile strength to modulus of rupture
1000	230	110	23.0	11.0	48
2000	375	200	18.8	10.0	53
3000	485	275	16.2	9.2	57
4000	580	340	14.5	8.5	59
5000	675	400	13.5	8.0	59
6000	765	460	12.8	7.7	60
7000	855	520	12.2	7.4	61
8000	930	580	11.6	7.2	62
9000	1010	630	11.2	7.0	63

*Data from tests made at Research Laboratory of the Portland Cement Assn., Chicago, Ill.

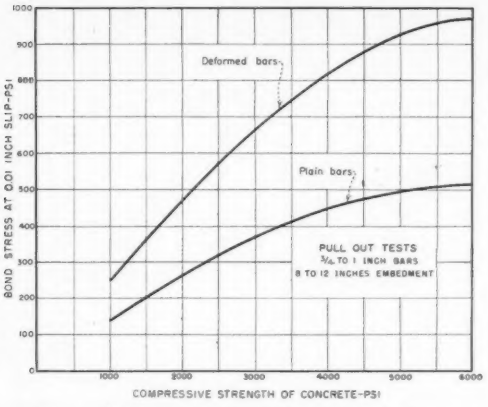


Fig. 16—Variation of bond with strength of concrete

RELATIONSHIPS OF 28-DAY CONTROL STRENGTHS AND CORES DRILLED FROM STRUCTURES

Table 3 shows the strengths obtained with cores drilled from the structure as compared to the 28-day strength of 6 x 12-in. control specimens. Note that the core strengths are almost all higher than the control strengths. These cores were from structures which had some curing, and it is expected that if cores were taken from columns of buildings which were never cured and were protected from moisture the cores might show lower strengths than the moist cured control specimens as indicated by Fig. 5.

LOSS IN STRENGTH DUE TO DETERIORATION

Concrete may start to deteriorate shortly after it is placed. Fig. 17 shows loss in strength resulting from cycles of freezing and thawing and alkali-aggregate expansion similar to the type of expansion occurring in Parker Dam. Because of this strength loss due to freezing and thawing, air-entraining concrete may be stronger after a few years than non-air-entraining concrete containing the same amount of cement, which was stronger at 28 days.

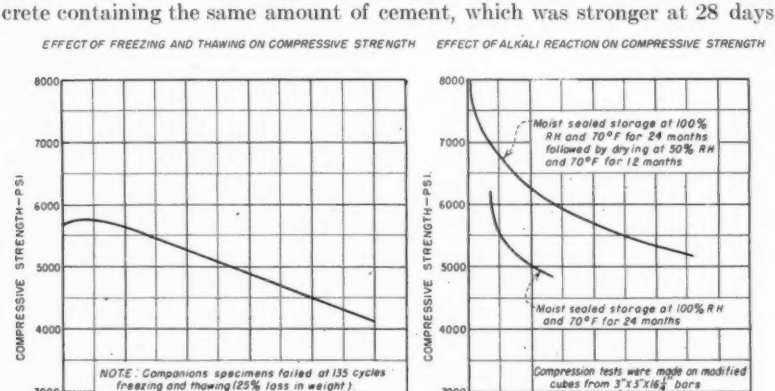

EFFECT OF FREEZING AND THAWING ON COMPRESSIVE STRENGTH
EFFECT OF ALKALI REACTION ON COMPRESSIVE STRENGTH
COMPRESSIVE STRENGTH—PSI
NOTE: Companions specimens failed at 135 cycles freezing and thawing (25% loss in weight)
All specimens were fog cured 180 days before start of test. 3" by 6" cylindrical specimens.
CYCLES OF FREEZING AND THAWING
Moist sealed storage at 100% RH and 70°F for 24 months followed by drying at 50% RH and 70°F for 12 months
Moist sealed storage at 100% RH and 70°F for 24 months
Compression tests were made on modified cubes from 3"x3"x16¼" bars
PERCENT EXPANSION AFTER 24 MONTHS MOIST STORAGE

Fig. 17—Loss of compressive strength due to deterioration

TABLE 3—COMPRESSIVE STRENGTH OF CONCRETE CORES AND CONTROL CYLINDERS

Project and feature	Cement type	Core diam, in.	Average age, months	Mix data: Cement, bbl per cu yd	Mix data: Net W/C by weight	Mix data: Maximum size aggregate, in.	Mix data: Slump, in.	Core strength, lb per sq in.	Number of cores tested	Coef. of variation, percent	Control cylinder, psi*	Strength ratio, core/cylinder, percent
Mass concrete—Large diameter cores												
Columbia Basin, Grand Coulee Dam	Modified	22	26	1.00	0.57	6	2	7220	6	5	5500	131
TVA, Wheeler Dam	Modified	18	16	1.15	0.53	3	1	6120	12	15	4570	134
TVA, Norris Dam	Modified	17	6	1.02	0.67	6	½	4880	6	8	4230	115
Columbia Basin, Grand Coulee Dam	Low-Heat	22	21	1.00	0.54	6	2½	7910	6	3	4640	172
TVA, Hiwassee Dam	Low-Heat	17	8	0.85	0.80	6	½	3800	6	9	2640	144
Central Valley, Shasta Dam	Low-Heat	22	12	1.00	0.61	6	1½	5430	3	3	3165	172
Central Valley, Shasta Dam	Low-Heat	22	24	1.00	0.61	6	1½	5450	3	7	3165	172
Central Valley, Shasta Dam	Low-Heat	22	60	1.00	0.61	6	1½	4920	3	12	3165	156
Central Valley, Friant Dam	Low-Heat	22	6	1.00	0.56	6-8	1	5230	2	1	5830	90
Central Valley, Friant Dam	Low-Heat	22	24	1.00	0.56	6-8	1	5750	3	3	5830	99
Central Valley, Friant Dam	Low-Heat	22	60	1.00	0.56	6-8	1	5630	3	1	5830	97
Mass Concrete—6-in. diameter cores												
Salt River, Horse Mesa Spillway tunnel	Standard	6	24	1.24	0.57	3	2½	5270	2	12	5060	104
Columbia Basin, Grand Coulee Dam	Modified	6	7	1.00	0.53	6	2	4300	7	11	5950	72
All-American Canal, Imperial Dam	Modified	6	34	1.21	0.58	3	3½	4770	6	16	3650	131
Columbia Basin, Grand Coulee Dam	Low-Heat	6	9	1.00	0.53	6	2¼	4850	28	16	4490	108
Columbia Basin, Grand Coulee Dam	Low-Heat	6	21	1.00	0.54	6	2¼	7330	15	10	4640	158
Salt River, Bartlett Dam	Low-Heat	6	15	1.24	0.58	3	3½	5840	16	13	3180	183
TVA, Hiwassee Dam	Low-Heat	6	8	0.85	0.80	6	½	4250	33	12	2640	161
Other than mass concrete—4- to 6-in. diameter cores												
Boise, Payette Division, Canal structures	Standard	4&6	3	1.41	0.58	1½	4½	4080	11	13	3480	117
Owyhee, Canal structures	Standard	4	19	1.40	0.55	1½	2½	4450	8	7	3640	122
Salt River, Horse Mesa Spillway tunnel	Standard	6	24	1.37	0.57	1½	2½	5490	4	3	4550	120
Boise, Payette Division, Canal lining	Fine std	4	6	1.40	0.57	1½	2½	5980	20	12	4330	135
All-American Canal: Imperial Dam; Canal structures and Imperial Dam	Modified	6	31	1.45	0.55	1	4	5460	4	15	3820	153
	Modified	6	25	1.38	0.55	1½	3¼	5970	133	15	4000	149
Columbia Basin, Grand Coulee Dam	Modified	6	7	1.30	0.53	1½	3	4250	7	7	6080	70
Gila, Canal structures	Modified	6	19	1.40	0.58	1½	3½	4390	43	16	3640	120

*6 x 12-in. specimens tested at 28 days

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Title No. 47-32

How to determine the condition of a concrete structure in the field has long been a problem. The Soniscope is a promising aid in such studies.

Soniscope Tests Concrete Structures*

By E. A. WHITEHURST†

SYNOPSIS

The Soniscope, an instrument which measures group velocities through as much as 50 ft of concrete, was used for the field testing of 13 bridges, one navigation lock, 14 dams and five highway pavements in 12 states. Repeated tests permitted study of changes in the condition of the concrete and the development of group velocities indicating the condition of the structure. The value of the results increases with knowledge of the materials, mix design, method of placement and other characteristics of the structure being tested.

INTRODUCTION

Since 1947, the Portland Cement Assn. has been applying dynamic testing techniques to concrete structures in the field. The approach selected was that of measuring the group velocity of small mechanical vibrations passing through the concrete.

Some work in this field was reported in 1945 by Long, Kurtz and Sandenaw.[1] More recently, an instrument known as the Soniscope has been developed by the Hydro-Electric Power Commission of Ontario. The Soniscope can measure group velocities through as much as 50 ft of concrete and on specimens of practically any shape. This work has been reported by Leslie and Cheesman.[2]‡

Through the courtesy of the Hydro-Electric Power Commission of Ontario the Portland Cement Assn. was permitted to make a study of the Soniscope and to build similar equipment for its own use. Since late in 1947, when the Association completed its first Soniscope, tests have been made on 13 bridges, one navigation lock, 14 dams, and five highway pavements. These structures are in 12 states, largely along both coasts and in the southwestern part

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‡The soniscope has also been used to study the rate of hardening of concrete at early ages, that is, during the first 24 hours. A paper by Arndt[3] describes such tests on pavement, and a paper by Jones[4] describes tests of laboratory specimens at early ages.

of the United States. In addition, group velocity tests have been made on seven projects of the Long-Time Study of Cement Performance in Concrete.

All personnel and the equipment, including a portable power supply, were easily transported in a station wagon. Three men made the tests: two handled the transducers, and one operated and took readings on the associated electronic apparatus.

Since the method and instrument have been thoroughly discussed by Leslie and Cheesman,[2] a similar discussion will not be repeated here. Briefly, the technique involves the measurement of the transit time of a minute mechanical vibration which is passed through the concrete under test. The path length between the transmitting and receiving transducers is measured, and the velocity of pulse propagation computed.

INTERPRETATION OF TEST RESULTS

In analyzing data from group velocity tests no effort has been made to convert values of group velocity to Young's modulus of elasticity. In most cases the tests have been made with the intention of repeating them at a later date to determine if any changes to the concrete were occurring in the particular structure involved. To use group velocity as a direct measure of quality, the following tentative classification has been established experimentally for normal concrete having a unit weight of approximately 150 lb per cu ft.

Group velocity, fps	Condition
Above 15,000	excellent
12,000-15,000	generally good
10,000-12,000	questionable
7000-10,000	generally poor
Below 7000	very poor

These lines of demarcation cannot be sharply drawn, exceptions being noted in all but the extreme classifications. They are based upon measurements through uncracked concrete in which continuity is not affected by joints.

In some exposures it has been found that group velocities of certain specimens are lower immediately after a severe winter than they are in the same specimens at the end of the following summer. The increase during the summer months is relatively large for concrete of low group velocity and small for concrete of high group velocity. Under such conditions the concrete normally shows a drop in group velocity during the winter comparable in magnitude with the summer gain. This indication of the recuperative powers of concrete may be similar to that found in laboratory freeze-thaw specimens in which both sonic and group velocity tests indicate marked, though temporary, improvement with moist storage following the freeze-thaw exposure. Resumption of the freeze-thaw test quickly reduces the specimens to the condition existing prior to the moist storage. In consequence of these effects the quality of concrete may, under certain conditions,

be either better or poorer than the group velocity might indicate. For laboratory specimens under accelerated freezing and thawing tests the discrepancy may be appreciable; on field structures in good condition it probably would be small.

TESTS ON BRIDGES

Most of the bridges tested to date are in the southeastern coastal states. One of these, in Georgia, is a relatively new structure shown in Fig. 1. The major portion of the substructure was built with natural sand, a potentially reactive gravel and a cement with more than 0.6 percent alkali. Crushed granite was substituted as coarse aggregate in the remainder of the concrete. Group velocity tests were made on this structure in 1948 and again in 1949 in an effort to discover any early evidence of deterioration. These tests were made on the first two piers appearing in Fig. 1 and on the abutment at the same end of the bridge. A total of 25 tests were made each year, the test locations in 1949 being idential with those in 1948. Tests were made through the pier shafts, the pier caps, the web walls and combinations of these elements. Path lengths varied from 15 in. to slightly more than 30 ft. A light scaffold and considerable rigging were required to reach some of the points of test. The scaffold may be seen in Fig. 1 as it was being moved into position at the third pier.

The largest difference in velocity between 1948 and 1949 measurements at any point was 830 fps. The average of 17 tests of the portion containing gravel was 13,690 fps in 1948, and 13,730 fps in 1949, a difference of 40 fps. The average of three tests of the portion containing granite was 12,320 fps in 1948, and 12,340 fps in 1949, a difference of 20 fps. That these results checked so closely was particularly gratifying because the Soniscope model used in 1949 was not the same as the one used in 1948. Five tests were made through the cap of the second pier in Fig. 1. Path distances varied from 4 to 30 ft; velocities ranged from 13,640 to 13,540 fps.

Fig. 1—Highway bridge in Georgia

It is perhaps well that this was one of the first structures tested, as it illustrated forcefully the necessity of knowing as much as possible about any concrete undergoing test before trying to interpret group velocity data. Velocities in the two piers ranged from 14,200 to 13,200 fps; those in the abutment from 12,390 to 12,290 fps. If no further information were available it might be concluded that concrete in the abutment was of a lower quality than that in the piers. Examination of the construction records established, however, that the gravel was used in the piers and the crushed granite in the abutment. The average difference between velocities in the piers and those in the abutment, roughly 1400 fps, is comparable to the difference found in specimens made from similar aggregates on the Georgia Experimental Farm of the Long-Time Study of Cement Performance in Concrete. Thus it appears that differences in group velocity found between the piers and one abutment of this structure are due to the materials used in construction and are not necessarily indicative of changes in properties of the concrete.

Two other bridges tested in this same vicinity have shown little change in a year. At the time of the original tests both structures showed considerable pattern cracking. Before the second series of tests were made both structures were repaired and treated with a waterproofing paint in an effort to prevent further cracking. However, group velocities as low as 11,020 fps in some members of one of these bridges suggests that some deterioration had developed before the structure was treated and that these members should be examined periodically for signs of further progress in the deterioration.

In June, 1949, an interesting series of tests was made on a bridge in eastern North Carolina. This bridge is supported by more than 300 bents, each composed of three 16 or 18-in. octagonal concrete piles and a concrete cap. Nearly all of the more than 1000 piles show some evidence of deterioration near the water level. In most cases this is confined to rather slight etching but about 250 show signs of more advanced deterioration with some surface spalling. About 50 piles have suffered severe deterioration with concrete spalled to a depth of 2 or 3 in. exposing the reinforcing steel. Fig. 2 shows one of the most severely affected piles. It has been suggested that the damaged piles might be reclaimed by chipping away the deteriorated concrete and casting collars around the sections to be repaired. The problem then is one of determining the extent of the deteriorated area.

At the time these tests were made the water level was approximately 5 ft below the bottoms of the pier caps. Highway engineers familiar with the structure stated that this was about 2 ft above the normal level. Two of the piles had been encased in caissons and could be examined and tested from cap to river bottom. Table 1 shows the results of group velocity tests on these piles, one of which was badly deteriorated.

These data indicate that the deteriorated area extends further above than below the water level (taken as approximately 7 ft below the caps). A number of the other piles were tested from water level to cap. In a few of the

TABLE 1—GROUP VELOCITIES THROUGH BRIDGE PIERS

Pier 301, center pile		Pier 280, center pile	
Distance below cap	Group velocity, fps	Distance below cap, ft	Group velocity, fps
2 ft	13,400	3	13,600
3 ft	12,500	5	12,800
4 ft	6,700	7	13,300
4 ft 5 in.	no signal	9	13,600
6 ft 5 in.	10,600	11	13,600
8 ft 2 in.	13,400	13	13,600
9 ft	13,600	15	13,600
11 ft	13,600		

Fig. 2—Deterioration in concrete pile—North Carolina

more severely damaged piles velocities increased uniformly from water level to cap. In most of the deteriorated piles velocities increased from water level to an elevation about 3 ft below the cap and remained constant for the upper 3 ft. Tests on a few of the piles which were only slightly etched showed the group velocity to be essentially constant over the entire exposed length. These data suggest that the majority of the piles are sound and that for most of the damaged ones a 5 ft collar extending from about 3 to 8 ft below the cap would cover the deteriorated section. A few of the more severely damaged piles might require more extensive repairs. Fig. 3 shows the transducers in place for testing the piles.

In an effort to obtain some data for comparative purposes a limited number of tests were made on four similar bridges in this area. One of these, several miles upstream from the bridge described in the preceding paragraphs, was supported on pile bents comparable with those shown in Fig. 3. Velocities measured through these piles ranged between 14,600 and 13,600 fps. In every case the velocity near water level was 200 to 600 fps lower than that at the top of the pile. None of these piles showed visible indications of deterioration other than slight etching of the concrete near water level.

The first structure in the United States to be tested with the Soniscope (June, 1948) was a heavily travelled bridge in Illinois. Results of tests on this bridge serve as an excellent example of the need for caution in interpreting tests of this nature. There were areas on the under side of this structure and its supporting members representing all stages of deterioration. Tests

Fig. 3—Group velocity tests—Highway bridge in North Carolina

on major members showed the wave velocity to vary from 15,830 to 5170 fps. Most of the velocities measured were well below 10,000 fps. Within a year after these tests all traffic beneath the bridge had been banned and the need for major repairs was clearly indicated.

These tests are not reported here solely to show the ability of the Soniscope to detect the presence of deteriorated concrete in this structure. A casual inspection would have shown that portions of the structure were in distress. The important fact is that, even in the case of a structure in such poor condition, large areas of concrete could be found which were in good condition. Conversely, it has been found that in structures of generally good condition, areas may sometimes be found which are indicated by group velocity tests to be of poor quality. Thus it can be seen that spot tests, poorly chosen as to location, could have indicated that this Illinois bridge was in good condition, just as similar tests on apparently sound structures might lead to the erroneous conclusion that the structures are in poor condition.

Experience has shown the desirability of first inspecting the structure to be tested and deciding upon a number of test positions which are likely to represent the extreme range in the quality of the concrete. If any of the differences in velocities measured at these positions are large, further tests must be made to determine the prevalent condition in the structure and the extent of the concrete which differs from this condition. Fortunately the tests may be made rapidly, as many as 80 having been made on one bridge in a day and a half.

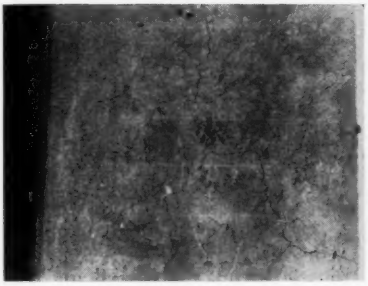

Fig. 4—Spillway gate superstructure—Arch dam in southwestern Arizona

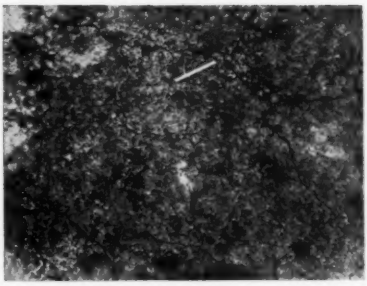

Fig. 5—Walkway—Arch dam in southwestern Arizona

TESTS ON DAMS

In the spring of 1949 group velocity tests were made on a number of dams in the southwest. One of these, an arch dam in western Arizona, had been reported as showing rather severe pattern cracking in the gate superstructure, walkways and pavement over the dam. Fig. 4 shows this condition in a typical member of the superstructure; Fig. 5 shows a section of the walkway. The cracking in the superstructure tends to form large patterns with a preponderance of vertical cracks. All crack openings are about $\frac{1}{64}$ in. In the walkways the cracks form a pattern of about 1 ft dimensions. The scale shown in Fig. 5 is 6 in. long. Most of these cracks are open $\frac{1}{16}$ to $\frac{1}{8}$ in.

A number of group velocity tests were attempted on these portions of the dam. Few were successful. Full power, usually sufficient to pass a vibration through 50 ft of concrete, was required to test through 8 to 10 ft in this case. Velocities through the gate superstructure averaged about 7500 fps. Those through the walkway were approximately 10,000 fps. The question had been raised as to whether the cracks in this structure extended far into the concrete or were confined to a very shallow depth. It was the opinion of those conducting the tests that the cracking extended to a depth sufficient to affect the group velocity determinations. There was no evidence to show that the low velocities were due to any uniform deterioration throughout the concrete. Rather, it is believed that the cracks are extensive enough within the structure to intercept the test vibrations. This would mean that the vibration finally received from the transmitter had travelled over a devious path rather than along a straight line. Thus the path traversed would be indeterminate but would in any event be longer than the straight-line path normally followed. When the measured transit time for this longer unknown path is used with the measured length of the straight-line path to compute velocity, the result will be a computed velocity which is lower than the correct velocity by some indeterminate amount. Nevertheless, such tests may be of value as an indication of whether or not the cracking is of an extensive nature.

At the time these tests were made, similar tests were made through the lower levels of a powerhouse near the dam. Velocities measured varied from

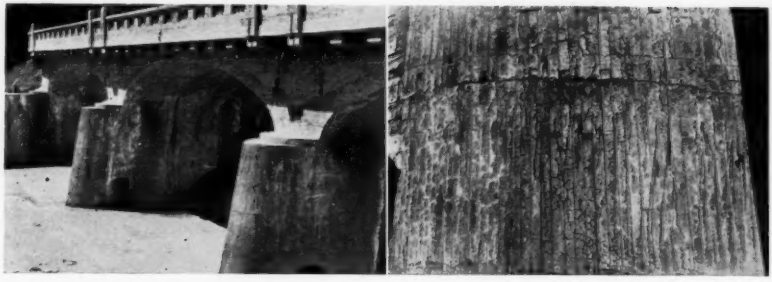

Fig. 6—Left—Spillway of dam in eastern Arizona. Right—Severe cracking in pier of left spillway

13,000 to 16,000 fps. In some positions, especially near the draft tubes, testing was very difficult because the normal vibration of the structure was much greater than the test vibrations. Test paths longer than 15 ft were not attempted.

Another dam upon which tests were made is in eastern Arizona. Several stages of deterioration were noted in the concrete of this structure. Concrete in the piers of the left spillway showed extensive and rather severe cracking, most of it forming a small open pattern. Fig. 6 shows two views of this spillway, one a general view and the other a close-up of one pier. The right spillway appeared to be in much better condition. A few cracks have opened in the upper parts of the piers but the lower portions are free from cracking. Fig. 7 shows a close-up of concrete in this spillway. The buttresses of the main structure showed no signs of cracking and appeared to be in excellent condition.

Group velocities measured through the piers of the left spillway averaged 10,900 fps. In the right spillway, measurements through the upper portions of the piers averaged 12,000 fps while those through the bottoms averaged 13,200 fps. Tests through the buttress showed the group velocity to be about 14,300 fps. Unfortunately, it may not be presumed that the highest velocities approximate those throughout the structure before any deterioration occurred and that the difference between these and lower ones indicates quantitatively the degree of deterioration which has occurred. It is quite possible that there were initial differences between different sections of the structure. On the basis of the previously stated general classification of quality versus group velocity, however, it would appear that the concrete in the buttresses is very good, that in the bottoms of the right spillway piers good, that in the tops of these piers approaching the questionable range, and that in the left spillway piers definitely in the questionable range perhaps approaching poor.

A third large dam tested is in Nevada. In this case the Soniscope and its associated equipment were carried down into the dam and tests made on the concrete in the center of the structure. Such tests are desirable when facilities

Fig. 7—Moderate cracking in pier of right spillway

are available for making them, as it has been found that the condition of the concrete forming the mass may differ considerably from that forming the shell. A series of 7-ft grouting tunnels had been constructed circumferentially through this dam. The vertical distance from the roof of one tunnel to the floor of the other was 43 ft. A number of tests were made between pairs of adjacent tunnels. The average velocity measured was 15,500 fps, with a minimum of 15,300 fps.

Other tests were made inside the dam, through outside walls and through spillway piers. About one-fourth of the measurements showed velocities in excess of 16,000 fps, with a maximum of 17,500 fps. The minimum velocity found was 14,800 fps, which was measured through a 15-in. parapet wall around the roof of the powerhouse. The average of all tests made on this dam and associated structures was 15,700 fps.

From these tests it may be concluded that the concrete in this dam is in excellent condition. It may also be concluded that the horizontal lift joints were tightly closed. About 10 such joints occurred in the path of the test signals between grouting tunnels. A small discontinuity at any of these joints would have made the received signals too weak to be readable.

Tests on another dam are interpreted to indicate that some of the lift joints were partially open. This structure, a multiple arch dam, is in central California. The arch barrels are approximately 18 in. thick and the sections tested have been covered with about 2 in. of pneumatically-placed concrete in the upstream face. Along the centerline the face of the arch is inclined about 30° from the vertical with the angle becoming less at the haunches where adjacent arches join. The lift joints are horizontal. Velocity tests were made in a direction perpendicular to the face of the arch, or about 30° from the horizontal. Thus, in the vicinity of a lift joint, one transducer would be below the lift line and the other above, with the joint intercepting the signal path.

Tests were made at frequent intervals along the centerline of one arch, starting at the bottom and progressing to the crest. As long as both transducers were against concrete in the same lift, velocities were found to average 12,400 fps. It was found in several cases that as one transducer was moved to the lift above, the other remaining against the lower lift, the computed

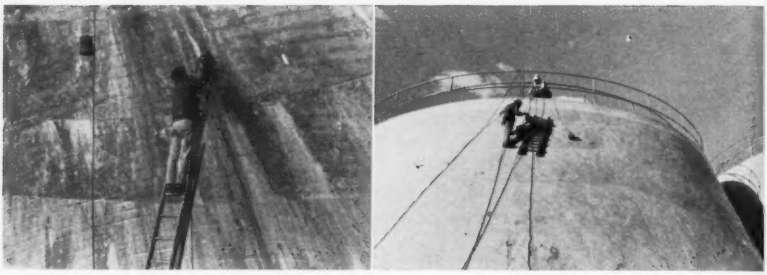

Fig. 8—Left—Transmitting transducer on underside of multiple arch barrel. Right—Receiving transducer on upper face of multiple arch dam

velocity fell to about 9000 fps, and much greater power was required to make the test. Fig. 8 shows two views of the testing operations, one depicting the transmitting transducer against the downstream face and the other showing the receiver against the upstream face of the arch.

These results clearly indicate that the lift joints are at least partially open. Instead of travelling a straight path between transducers, the test vibration must have followed some longer route to reach the receiver. This path may have extended around the end of the joint and through the pneumatically-placed concrete or it may have extended through some closed portion of the joint which was not directly on the line between transducers. The fact that velocities in general are rather low (12,000-13,000 fps) is not viewed as being indicative of excessive deterioration. Since the original mix is reported to have had a very high water-cement ratio, initial velocities probably were low.

TESTS ON HIGHWAYS

Group velocity tests have been made less extensively on pavements than on other concrete structures, and have not covered as wide a range of conditions.

In making these tests on pavement slabs the following procedure has generally proved satisfactory. A small hole is scooped in the shoulder, exposing one edge of the slab. The transmitting transducer is placed in firm contact with this exposed face. The receiving transducer is then held against the surface of the slab at some known distance from the transmitter. Fig. 9 shows the transducers in place for such a test on a pavement in New York. It is good practice to move the receiver so that paths of several different lengths are traversed while the transmitter is left in one position. If all of the computed velocities are approximately the same in spite of the variations in path length, there is every reason to believe the computed velocity is correct. If the velocities vary appreciably it is probable that something—possibly very fine cracks—is interfering with the test signal, and results of such tests should be used with caution.

Tests were made on two sections of pavement in North Carolina in July, 1949. One of these was a new pavement only 13 days old at the time tests

Fig. 9 — Testing highway slab concrete in New York

were made and not yet open to traffic. Velocities measured ranged from 13,800 to 14,200 fps with an average of 14,050 fps. On the same day velocities were measured through two test beams cast from concrete used in the sections of road on which tests were made. Velocities through these beams were 13,900 and 14,000 fps.

The second section of highway tested in North Carolina was built in 1936. The coarse aggregate used was one which has since been rejected on the basis of the Los Angeles Rattler Test. Group velocities measured ranged from 15,300 to 14,600 fps, and are reported here because these are among the highest yet measured in highway pavements.

In October, 1949, tests were made on a freeway in California. Group velocities were measured in concrete containing no entrained air and in concrete to which an air-entraining agent had been added at the mixer. Different amounts of this agent were used in two sections of the pavement. Velocities through the concrete without entrained air averaged 15,300 fps; those measured in the concrete containing the least amount of air-entraining agent averaged 15,100 fps; and the average velocity in the section cast with the larger amount of air-entraining agent was 14,800 fps. No visual evidence of deterioration could be found. The results of these tests agree with similar tests on laboratory mixed concrete intended to show the effect of air entrainment on group velocity.

SUMMARY

The results of two years experience in the measurement of group velocity through concrete indicate clearly that this technique is applicable to studies of the performance of concrete in field structures. The technique, however, is recognized to have limitations,* and the information gathered with this tool should be used in conjunction with all other available data in reaching conclusions concerning the condition of any structure tested. Although this

*Basically, the Soniscope measures but one property of concrete and the haphazard interpretation of other properties will lead only to incorrect conclusions. Technical limitations of the instrument would require a discussion of electronics and transducer design far beyond the scope of this paper.

type of test will rather definitely identify concrete of very good or very poor quality, the more complete the knowledge of concrete mix proportions, aggregates, methods of placing, etc., the greater the assurance which may be placed upon the interpretation of group velocity measurements. Properly interpreted, changes in group velocity with time appears to have particular significance in studying the performance of concrete, increasing group velocity indicating improvement in quality, decreasing velocities indicating deterioration.

The Soniscope, as presently designed and constructed, is portable, easily operated and capable of measuring velocity through sections varying from 2 in. to 50 ft. It is probable that further experience may lead to improved design which will extend the maximum range.

The work to date has been largely confined to studies of group velocities in concrete, but it may be stated that very limited laboratory experiments have shown that the same method and instrument may be used to test soil-cement bases, bituminous macadam pavements and rock cores, and perhaps to indicate changes in compaction and moisture content of soil subgrades. The extent of these investigations is too limited, however, to do more than to indicate strongly the desirability of further exploration of these fields.

ACKNOWLEDGMENTS

The author is indebted to J. R. Leslie and W. J. Cheesman, Hydro-Electric Power Commission of Ontario, Toronto, for their assistance and advice during the early stages of the development of the Soniscope and for their continuing cooperation in studying and analyzing test results, and to representatives of the state highway departments, governmental agencies and private organizations who helped make these tests possible.

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Discussion of a paper by E. A. Whitehurst:

Soniscope Tests Concrete Structures*

By EDWIN J. CALLAN and AUTHOR

By EDWIN J. CALLAN†

The author is to be commended for his excellent paper on the results of field use of this valuable tool for concrete research. The sound-velocity methods should in time supplant the resonant frequency method for determining elastic constants of concrete due to their much wider range of application. For field structures they will prove practically indispensable.

It was the privilege of the writer to see the method in operation in the summer of 1948, and to be able to compare the two methods, sound-velocity and resonant frequency, as applied to mass concrete of a structure. The comparison is of interest in several ways.

The structure was built between 1937 and 1939 and has exhibited expansion and cracking in several of the monoliths. In the course of an investigation of the causes of this cracking a 36-in. calyx core was drilled vertically through one of the monoliths at a distance of several feet from the vertical surface. Prior to drilling the 36-in. core, a 6-in. core was diamond-drilled through the center of the calyx core site and sawed into sections. The sections were tested for dynamic moduli of elasticity, both Young's modulus and modulus of rigidity, using standard methods for determining resonant flexural and torsional frequencies.

Shortly after this was done, a demonstration of the sound-velocity method was undertaken at this structure under the direction of Messrs. I. L. Tyler and E. A. Whitehurst of the Portland Cement Assn. using equipment similar to that of the Hydro-Electric Power Commission of Ontario. Mr. J. R. Leslie of the Hydro-Electric Power Commission of Ontario was present as consultant. In the course of the demonstration, measurements were taken of wave-velocities at various depths in this particular monolith, using the vertical surface of the structure and the wall of the 36-in. core hole as locations for the transducer and receiver. Table A compares Young's modulus of elasticity of the concrete at various depths as derived from the resonant frequencies and from the sound velocities. The modulus of elasticity was obtained from the sound velocity readings by computing values from the following equations, based on a massive structure:

*ACI JOURNAL, Feb. 1951, *Proc.* V. 47, p. 433. Disc. 47-32 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

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TABLE A—COMPARISON OF YOUNG'S MODULI DETERMINED BY SOUND-VELOCITY AND FLEXURAL FREQUENCY TESTS

Depth in monolith, ft		Longitudinal velocity, fps	Young's modulus, $E \times 10^{-6}$, psi	
Sound-velocity	Frequency		Sound-velocity	Frequency
5.0	0.0-1.5;6.0-7.5	12,200	3.9	2.0
8.5	7.5- 9.0	13,890	5.3	4.3
12.0	11.0-12.5	14,390	5.8	5.5
16.0	18.9-23.8	14,610	6.0	5.5
20.0	18.9-23.8	14,890	6.1	5.5
25.0	23.8-28.8	14,910	5.8	5.9
29.0	28.8-33.5	14,810	6.1	5.5
33.0	28.8-33.5	14,930	6.2	5.5
35.0	33.5-39.6	15,110	6.2	5.8

$$E = \frac{KV^2P\,(1 + \mu)\,(1 - 2\mu)}{(1 - \mu)}$$

where:

E = modulus of elasticity, psi
V = sound velocity of longitudinal wave, fps
P = density of concrete
μ = Poisson's ratio of concrete (taken as 0.25)
K = numerical factor for conversion of units

This simplifies to

$$E = \frac{WV^2}{5564.16}$$

where:

E and V are as above
W = unit weight of concrete, lb per cu ft

The comparison between values for Young's modulus obtained by the two methods is quite good, and for all readings except those within 10 ft of the surface, the moduli for comparable depths agree within 10 percent.

The moduli for sections within 10 ft of the surface however show differences up to 90 percent; with those computed from the velocity measurements being higher than those computed from resonant frequencies. A possible explanation for this lies in the fact that the concrete near the exposed surface of the monolith is characterized by considerable fine cracking. Hairline cracks are sufficient to destroy or markedly reduce structural homogeneity of concrete and thus reduce the modulus of elasticity as determined by frequency measurements. At the same time they are not sufficiently large to appreciably affect the transit time of the transmitted wave, the sound wave being transmitted across the fine cracks, and thus a relatively high modulus is maintained, as computed from the wave velocity. In such cases, the use of the sound-velocity measurements may lead to higher evaluation of the quality of the concrete than should be the case. To avoid this, perhaps the use of equipment to measure attenuation, using the power dissipation as a

measure of the soundness of the concrete, would be of assistance in conjunction with the sound-velocity measurements.

It is believed that values for Young's modulus should be computed from sound velocity data even though, as indicated above, some caution is required in their application. For many engineers, the value so obtained will be more readily appreciated than values for wave velocity or frequency.

AUTHOR'S CLOSURE

Mr. Callan, in his discussion, has reported some of the earliest field tests made with the Soniscope in the United States, and one of the few cases permitting a direct comparison between data from velocity tests and data from other tests performed in the laboratory upon the same concrete. The close agreement between the dynamic modulus of elasticity computed from the velocity measurements and that found by flexural vibration tests was most gratifying.

With respect to an explanation of the differences between these moduli in the top 10 ft of the structure, experience indicates that any actual crack, however small, is sufficient to cut off the transmission of the test vibrations. However, if the crack is not of very large lateral extent the vibrations will pass around it and eventually reach the receiver. In so doing, they have travelled a path somewhat longer than the presumed straight-line path upon which the velocity is computed. Thus the computed velocity will be lower than that through uncracked concrete in the same structure. The amount by which this velocity is erroneously low will be dependent upon the extent of the crack and its position with respect to the transducers. It is the opinion of the author that for concrete in the upper 10 ft of the structure under discussion the values of modulus of elasticity as determined by both methods of test are highly questionable. Results by both methods appear to have been affected by cracks in the concrete, the extent of this effect depending upon the random occurrence of the cracks in the specimen or section tested. It seems questionable whether the dynamic modulus of elasticity of concrete has any meaning, with respect to its quality, once the continuity of the test section—be it a small laboratory specimen or part of a structure—is damaged.

This brings up a final point which is worthy of brief discussion. Mr. Callan recommends that values of Young's modulus be computed from measured velocities, although he admits that some caution must be exercised in their evaluation. There may be cases, such as the one here mentioned, in which such a procedure is desirable to permit a comparison between field and laboratory tests on the same concrete. The author would have preferred making velocity tests on the cores and comparing velocities directly.

In actual practice, the absolute value of dynamic modulus is not of great worth. It is not used in design. It is probably a poor criterion for either compressive or flexural strength. It is widely used as a measure of progressive or relative deterioration. Even for this use, for which it seems highly

satisfactory, the engineer can scarcely draw valid conclusions from a study of the individual moduli of specimens representing different mixes or made from different materials. He must base his conclusions upon an evaluation of the differences in modulus for the same specimens over a period of time.

If Young's modulus is to be computed from field velocity tests a number of assumptions are required. Aside from the theoretical assumptions involved in the choice of a formula, concerning which there may still be some room for argument, values of density and Poisson's ratio must be assumed. In the majority of cases the engineer can only assume 150 lb per cu ft for the former and some constant value, perhaps 0.24 as suggested by Leslie and Cheesman,* for the latter. If this is done, then computed moduli will vary only as the velocity squared and are subject to such error as may be due to improper choice of density or Poisson's ratio. It would seem, then, that the velocity itself, a quantity which may be determined rather accurately, might be a better choice.

It is true that group velocity, as a measure of concrete quality, presently has little meaning to many engineers. It may be recalled, however, that only about a dozen years ago the now accepted dynamic modulus was equally new to the concrete field. The author is of the opinion that the need for a method of testing concrete in place is sufficiently great that engineers involved will soon accept the terminology applying to any technique satisfying this need.

*Leslie, J. R. and Cheesman, W. J., "An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures," ACI JOURNAL, Sept. 1949, *Proc.* V. 46, p. 17.

New materials and techniques increase the efficiency of joint seals in concrete pavements.

Effective Sealing of Concrete Pavement Joints*

By H. F. CLEMMER†

SYNOPSIS

The development of cork, metal and rubber joint sealers is traced and work with thermoplastic and cold-pour joint filling compounds is described. The importance of proper technique of preparing and placing the newer materials is emphasized.

INTRODUCTION

The maintenance of our roads and streets has become an enormous and important operation costing approximately two-thirds as much as new construction. Because existing pavements must be efficiently maintained to enable them to carry the greater loads imposed on them, highway officials and engineers are becoming aware of the need for scientific consideration of the methods and equipment used for this work. At the same time there is a realization of the need for inspection and periodic maintenance to provide positive and continuous sealing of the various types of joints and cracks in concrete pavements. A joint left open during only one prolonged rainy period may result in a serious loss of foundation support and ultimate failure of the concrete slab unless the soil base is rebuilt and the joint is sealed.

Experience has shown that it is difficult to satisfactorily seal a random crack which results from volume changes in a portland cement concrete pavement constructed without formed joints. For this reason, the practice of forming transverse joints with provision for load transfer is in general use by the major highway departments. Regardless of whether they are expansion or contraction joints, assuming the adequacy of load transfer design, the problem resolves itself into thoroughly sealing all joints against the infiltration of moisture and extraneous matter which may interfere with proper function of the formed joints. Because many sealing materials have been unsatisfactory, some engineers are now discouraging the use of expansion joints, except at wide intervals, as well as the installation and sealing of contraction joints.

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DEVELOPMENT OF SEALING MATERIALS

Joint sealers and joint designs have been primarily the result of initiative by the manufacturers. Many ingenious devices of presumably efficient materials have been proposed and used to a limited extent. Initial cost, practicability of handling and installing and ease of repair or maintenance are factors influencing the acceptability of such materials.

Almost since the advent of modern concrete pavement, many types and combinations of types of asphalt have been tried in an effort to secure a material fluid enough to pour into a crack or joint, sufficiently stable to resist the high temperatures of summer, sufficiently adhesive to bond to the face of the concrete of the joint and sufficiently ductile to stretch without failure with the contraction of the slabs. Various combinations of steam-refined asphalts, air-blown asphalts and mineral fillers have been used but none has proved entirely satisfactory. During warm weather, with slabs expanded, these materials extruded and flattened out on the surface of the pavement; in the following cold weather, when joints opened, the sealing material either pulled loose from the concrete or cracked down the center of the joint permitting the entrance of water and foreign material.

In an attempt to overcome the disadvantages of asphalt, the so-called self-sealing cork expansion joint was developed. The cork was dehydrated and sealed, usually with cellophane, against absorption of moisture until installation. The swelling properties of the dehydrated cork in air of average humidity were relied upon for maintaining a tight joint from top to bottom in all weather. Although the cork provided excellent resiliency without extrusion and was relatively durable, necessary handling frequently affected or destroyed the vapor-proof wrapper.

The M-shaped copper sealing device used with the metal air-cushion joint was developed and used in the early thirties. Although ideal from the design standpoint, the copper used in forming the M-shaped seal frequently failed to withstand the pressure and the repeated opening and closing of the joint. In addition, the necessary maintenance of such joints due to spalling and service cuts was found to be particularly difficult.

Vulcanized rubber fillers with integral sealers bearing wings for casting in concrete were developed on an experimental basis just prior to the war but further promotion has not been carried on (Fig. 1). A somewhat similar sealer which fits over any type of expansion joint material is, however, now available in detachable form (Fig. 2). Although considerable care is needed in the installation of these sealers, they warrant experiment and study.

Before World War II, the District of Columbia Highway Department used a rubber sealing compound made in its own laboratory. Crude rubber was softened with kerosene and mixed with resin, and the resulting mix stiffened and whitened with bauxite. This product proved satisfactory but was found to be nonuniform probably due to variation in the rubber resin used. Later a similar product was produced by one of the rubber manufacturers. In this case an emulsion of latex was used and on application was

Fig. 1 (Above)—Vulcanized rubber expansion joint integral premolded sealer

Fig. 2 (Right)—Concrete core cut across expansion joint showing a method of sealing with premolded rubber sealer

broken by the addition of an inert material, usually mica. Many joints filled with this material as long as ten years ago are still in good condition. There is lack of bond to the concrete but the material has retained its resilience to the extent that the joint space is completely filled during both low and high temperatures. The manufacture of this material was discontinued during World War II and latex for this purpose has not been made available since.

Hot poured, thermoplastic material, usually a mixture of bitumen and rubber, was developed and used extensively during the last war. When properly used this material bonds well to the concrete and has high recovery without significant extrusion. Field tests show definitely that satisfactory performance depends almost entirely on a uniform technique of handling, heating and controlling its temperature. Overheating was found to be particularly detrimental to full resilience. As a consequence, a commercially available "pouring unit" has been developed which insures proper handling through automatic temperature control, constant agitation, and a device for maintaining uniform pouring of the material along any given line. After years of service the resilience and stability of this sealing material is remarkable and far superior to most previous types. The excellent results obtained justify the rather high cost and the necessary apparatus and careful supervision.

Large quantities of this material have been used in airport and other construction with excellent success. Field surveys of joints in these concrete slabs indicate that if proper practice is followed, such as thorough cleaning and blowing out of the joint opening and prevention of overheating of the rubber sealing compound, efficient sealing of the joint may be expected.

From several viewpoints a cold-pour sealing material is desirable. Recently cold-pour materials which appear to have merit have been developed and are available for use. Laboratory and field tests indicate that these materials have advantages of low installation cost and can be applied under

moist conditions. Available data do not warrant definite conclusions, but it is believed a satisfactory material will eventually be perfected.

Although important developments have been made in improving joint sealing materials, a large percentage of our pavements have been constructed using sealing materials that have proved inadequate. Because of the use of material of questionable value, or perhaps because of the improper handling of otherwise satisfactory materials, there are many improperly sealed joints which should have careful inspection and constant maintenance. It is frequently found advisable to remove the old joint sealing materials entirely and reseal these joints. Recently, because of the realization of the importance of keeping joints properly sealed, machines have been developed which effectively cut out old joint material and scarify the adjacent concrete surface. Such power apparatus has been successfully used on a large number of airports, and has been found to be particularly adaptable to servicing straight expansion and contraction joints. It may also be used with some success in cleaning out irregular structural cracks which require sealing.

The first necessary step in rehabilitation of old expansion joints is to remove the old material and clean the concrete surfaces to insure proper bond of the seal material. The pouring of old joints not properly cleaned is a poor investment. Records of a recent cleaning and pouring of joints on a large airport indicated that approximately one third of the total cost was for labor, one third for cleaning (machine, etc.) and one third for seal material and equipment for application. Certainly the one third for cleaning to insure satisfactory service from the seal material is a good investment. Properly bonded seal material will last many years while a poor job may soon fail.

This logic applies to the use of a satisfactory seal material as well. A good seal may cost five times as much as an old type unsatisfactory seal, but the new type material will remain in place for five years or more as compared to the necessity of resealing the less durable joints each year. Further, the use of a satisfactory seal material with proper maintenance will insure complete seal of the joint throughout the year while the unsatisfactory material will probably not serve as a complete seal through a single change of season.

SUMMARY

Regardless of how good the soil base support is, maximum service will not be maintained unless *all* joints and cracks are completely sealed to prevent infiltration of moisture and foreign materials. The pavement slab must be made waterproof to protect the soil foundation.

Periodic and thorough maintenance of joint sealing is necessary to insure full life expectancy of concrete pavement. With increasing loads and traffic density, maintenance work must be carefully planned and executed on a definite engineering basis of equal importance to that of construction.

With study and care to effect a proper seal, inspection and maintenance keep joints that way.

These tests indicate that less protection against freezing during curing is required when air-entraining concrete is used.

Entrained Air Simplifies Winter Curing*

By JOSEPH J. SHIDELER, HAROLD W. BREWER and WILBUR H. CHAMBERLIN†

SYNOPSIS

The investigation described was undertaken to determine the winter protection required to protect air-entraining concrete from damage by freezing. Cylinders made with Types II and V cement and various percentages of calcium chloride were cured at temperatures ranging from 10 to 70 F and tested for strength at ages ranging from 3 to 180 days. The resistance of these concretes to accelerated freezing and thawing was compared. The results indicate that the amount of winter protection, as presently specified by the Bureau of Reclamation, can be reduced when air-entraining concrete is used.

INTRODUCTION

Present Bureau of Reclamation specifications covering winter protection of concrete require that the concrete shall be placed and maintained at a temperature above 50 F for not less than 72 hours, and that it shall be protected from freezing for 14 days for concrete made with Types I, II and V cement, and for 21 days for concrete made with Type IV or portland-pozzolan cements. In recent years, cores drilled from structures which were not protected for the specified period indicated that some shorter period of protection might be adequate, and, as this protection is costly, an investigation was initiated to determine whether it would be safe to reduce the amount of protection. Experience has shown that some protection is necessary during and immediately after the concrete is placed to prevent frost crystals from forming in the concrete.

This investigation was designed to determine the effect of freezing temperature on the strength and resistance to freezing and thawing of concrete at early ages. The initial temperature of all concrete was 50 F. Specimens were cast and cured at 50 F for various periods followed by storage at temperatures ranging from 10 to 70 F. The behavior of concrete made with

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Types II and V cement with 0, 1 or 2 percent calcium chloride was compared. All concretes used in this investigation contained approximately 4 percent entrained air. Nine hundred sixty 6 x 12-in. specimens were tested in determining the effect of low curing temperatures on strength, and two hundred seventy 3 x 6-in. cylinders were subjected to alternate cycles of freezing and thawing to determine the resistance of the concrete to this action.

The results of these tests show that no permanent damage is done to concrete made with Type II cement by freezing temperatures after 3 days of curing at temperatures above 50 F, provided moisture is available for hydration when the temperature is above freezing. The addition of calcium chloride to the mix decreases the time that heat must be furnished. As far as durability is concerned 1 day of curing at temperatures above 50 F is sufficient for concrete containing 1 percent calcium chloride, but 3 days protection are necessary to allow sufficient strength development. The strength developed by concrete placed at low temperatures is dependent on many factors; among which are the richness of the mix, the curing temperature, the type and brand of cement, and the percentage of calcium chloride used. The results of these tests show that, for concrete made with Type II cement and maintained at temperatures above 50 F, more than 50 percent of the 28-day strength is obtained in 7 days, and that when 1 and 2 percent calcium chloride is added to the mix, equal strengths are obtained in 4 and 3 days, respectively. Longer curing periods are required to obtain equal strengths for concretes containing Type V cement.

TEST PROCEDURE

The aggregate grading, mix data, and information on materials used in these tests are given in Tables 1 and 2. The materials were batched by weight, and were tempered and mixed in a room controlled to produce an initial concrete temperature of 50 F. Air contents were determined in a Klein-Walker air meter, and specimens were cast in open-top steel molds. It was found necessary to reduce the amount of Vinsol resin as the percentage of calcium chloride was increased to maintain the air content at approximately 4 percent. Otherwise, the mix was held constant. Aggregate graded up to

TABLE 1—MIX DATA

	Strength specimens	Durability specimens
W/C	0.50	0.50
Slump, in.	2	4
Cement,* lb per cu yd	476	600
Water, lb per cu yd	238	300
Aggregate, lb per cu yd	3250	2930
Percent sand	38	46
Percent air	4	4.5
Percent $CaCl_2$	0, 1, 2	0, 1, 2

*Cement—Type II cement, a blend of 10 commercial cements (Lab No. 8487); Type V cement, (Lab No. 9249)

TABLE 2—AGGREGATE GRADING*

Screen size	Cumulative percent retained: Strength specimens	Cumulative percent retained: Durability specimens
No. 8	15	15
No. 16	30	30
No. 30	55	55
No. 50	79	79
No. 100	95	95
Pan	100	100
FM	2.74	2.74
¾ to 1½-in.	45	—
⅜ to ¾-in.	78	60
No. 4 to ⅜-in.	100	100
FM	7.23	6.60

*Aggregate—Clear Creek gravel and Cherry Creek sand

1½-in. maximum size was used in the concrete for strength determination, and up to ¾-in. maximum size for the concrete used in the freezing and thawing durability tests.

After 24 hours, the molds were removed, and the 6 x 12-in. compressive strength cylinders were immersed in water maintained at 50 F for curing. The control specimens remained at 50 F until tested, while the others were given the following treatment after 3, 5 or 7 days of moist curing at 50 F: (a) stored in water at 40 F, or 32 F to 28 days' age, (b) frozen in air at 10 F for 3 days and returned to 50 F moist storage, or (c) frozen and thawed daily in air to 28 days' age and returned to 50 F moist storage.

Curing at 40 and 50 F was accomplished by placing cylinders in water baths in rooms controlled at the desired temperatures. Ice water was used to maintain temperatures at 32 F. Strength specimens subjected to mild freezing and thawing were frozen for 17 hours in a refrigerated room maintained at 10 F. They were removed from this room for a thawing period of 7 hours at 50 F and were sprayed with water just prior to being returned to the cold room. Temperatures obtained at the center of the specimens indicated complete freezing and thawing during this cycle.

Batch-to-batch variations were compensated for by careful dispersion of the strength specimens over the entire range of storage and testing periods.

The 3 x 6-in. durability cylinders were treated somewhat differently after removal from the molds. Some of the specimens were cured at 50 F for 28 days, while the others were cured at 50 F for 1, 3 or 7 days, then: (a) stored in a moist atmosphere at 10 F, 40 F or 70 F, (b) stored in dry air at 10 F, 40 F, 50 F or 70 F, or (c) given mild daily cycles of freezing and thawing for remainder of 28 days.

The mild cycles of freezing and thawing consisted of freezing in air at 10 F for 17 hours and thawing at 50 F for 7 hours. Part of the specimens were in air during the entire period while the other specimens were thawed in water.

These specimens were subjected to accelerated freezing and thawing tests at 28 days' age. They were placed in rubber containers, covered with water, and lowered into a specimen cabinet. Calcium chloride brines at 10 and 70 F were alternately circulated around the specimens in the cabinet every 1½ hours. Fifty cycles were obtained each week. Weight and length measurements were taken each week and the cycles were repeated until indications of quality were obtained.

COMPRESSIVE STRENGTH TEST RESULTS

Fig. 1a and 1b show the strength gain of specimens containing Type II cement, stored at 40 and 32 F after initial curing at 50 F. Fig. 1c shows the strength gain of specimens given initial curing at 50 F, then frozen for 3 days and returned to 50 F storage. The curves below these give results for concretes containing 1 and 2 percent calcium chloride. For purposes of com-

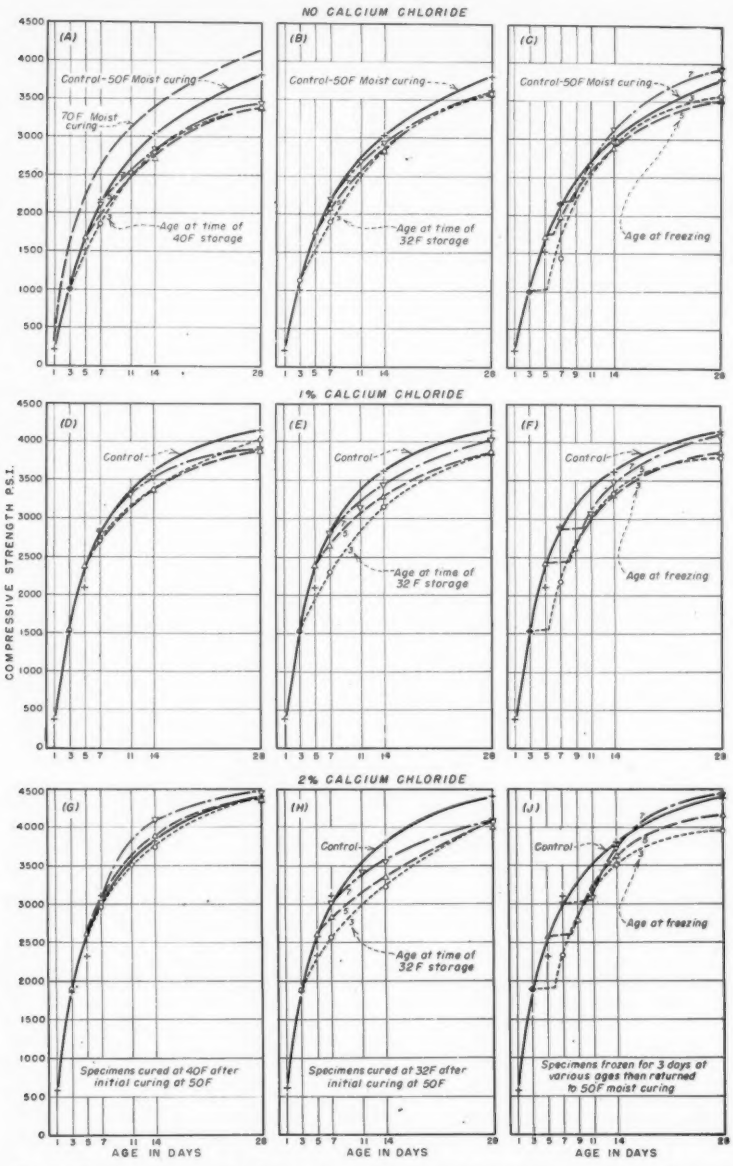

Fig. 1—Compressive strength of concrete cured at various low temperatures—Type II cement

parison, a curve which shows the compressive strength of specimens moist cured at 70 F has been added on Fig. 1a. Fig. 2 presents similar data for concrete containing Type V cement.

Freezing the concrete containing Type II cement for a period of 3 days after 3 or 5 days curing at 50 F reduced the 28-day strength about 7 percent, but freezing after 7 days had no effect on the 28-day strength. Storing the specimens at 32 F reduced the 28-day strength about 7 percent below the control strength, with no apparent difference between 3 and 7 days initial curing. The use of 2 percent calcium chloride raised the strength at all temperatures about 18 percent, and either 1 or 2 percent calcium chloride compensated for the low or freezing temperatures and produced greater strengths at all ages than the control without calcium chloride. Specimens without calcium chloride stored at 40 F after initial curing at 50 F had slightly lower strengths than the 50 F control specimens. Concrete with 2 percent calcium chloride stored at 40 F had strengths equal to that of the control specimens.

Fig. 2 depicts the age-strength relationship for specimens fabricated with Type V cement and indicates a greater loss of strength due to low curing temperatures than for specimens containing Type II cement. Freezing the specimens for 3 days at ages from 3 to 7 days reduced the 28-day strength about 13 percent. Storing the specimens at 32 F reduced the strength about 22 percent below the control strength. The effect of 2 percent calcium chloride was greater for the Type V cement and raised the strength at all temperatures approximately 30 percent. This increase in strength more than compensated for the losses due to the low or freezing temperature conditions. Specimens stored at 40 F after initial curing at 50 F had strength equal to those continuously cured at 50 F.

Fig. 3 shows the changes in compressive strength of concretes cured at 50 F for 3 days, then subjected to freezing and thawing for 25 days, followed by moist curing at 50 F. This period of freezing and thawing reduced the 28-day strength 30 to 40 percent for various amounts of calcium chloride with a Type II cement and 30 to 50 percent for the Type V cement. Freezing and thawing after 14 days protection reduced the 28-day strength 10 to 20 percent for both types of cement. Concrete with Type II cement and without calcium chloride subjected to 25 days of freezing and thawing after 3 days' protection, had the 28-day strength reduced 40 percent. However, when these specimens were returned to moist curing at 50 F, the strength increased rapidly and in less than a month exceeded the strength of the control specimen. This indicated that the strength development was only retarded by the low and freezing temperatures. The concrete containing calcium chloride did not completely recover from the effects of freezing and thawing.

The concrete containing Type V cement was retarded more by the period of freezing and thawing and recovered more slowly than concrete containing Type II cement. However, at age 180 days, the concrete with Type V cement

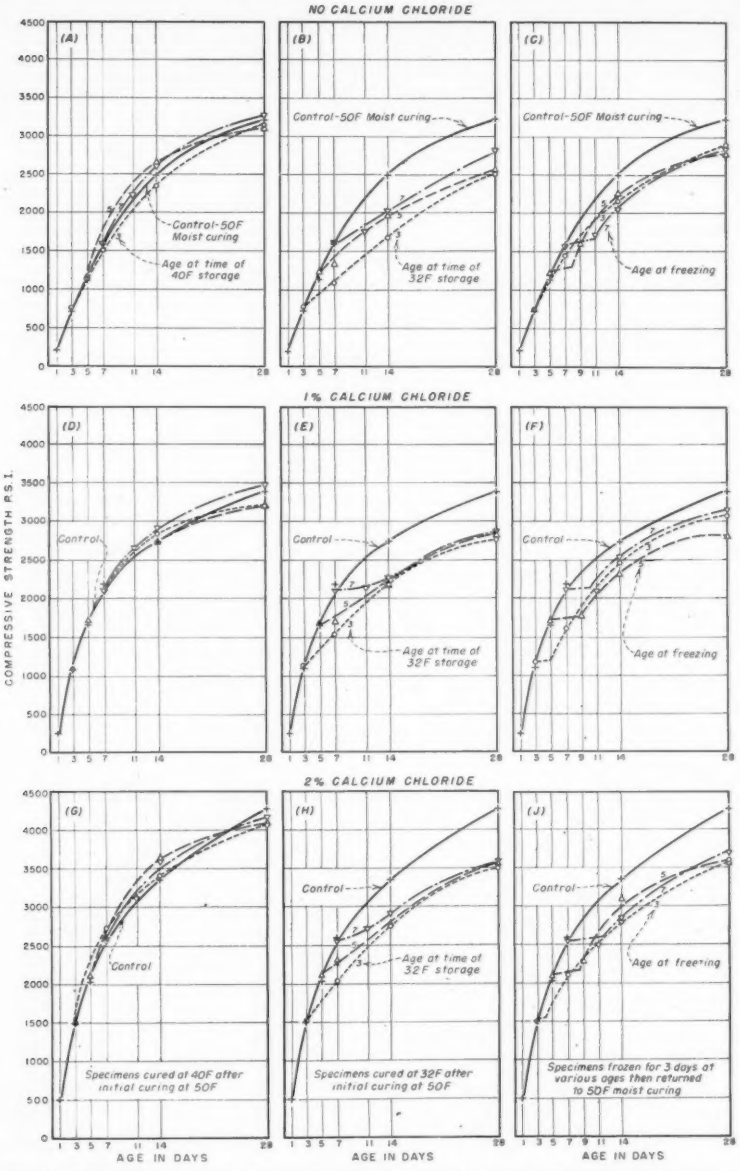

Fig. 2—Compressive strength of concrete cured at various low temperatures—Type V cement

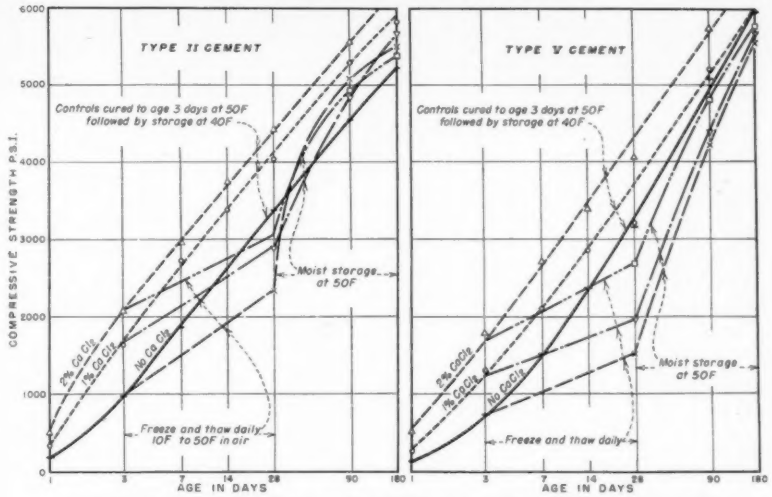
TYPE II CEMENT
TYPE V CEMENT
Controls cured to age 3 days at 50F followed by storage at 40F
Moist storage at 50F
Freeze and thaw daily 10F to 50F in air
2% Ca Cl2
1% Ca Cl2
No Ca Cl2
2% CaCl2
1% CaCl2
No CaCl2
COMPRESSIVE STRENGTH P.S.I.
AGE IN DAYS

Fig. 3—Compressive strength retarded by freezing and thawing at early ages

subjected to freezing and thawing had returned to within 7 percent of the control strength.

Fig. 4 illustrates the effect of calcium chloride on the compressive strength to age 28 days of specimens cured at low temperatures and containing Types

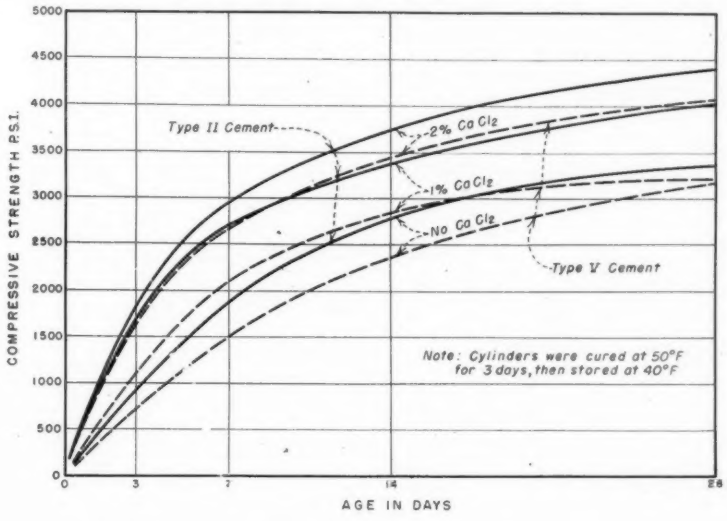
Type II Cement
2% Ca Cl2
1% Ca Cl2
No Ca Cl2
Type V Cement
Note: Cylinders were cured at 50°F for 3 days, then stored at 40°F
COMPRESSIVE STRENGTH P.S.I.
AGE IN DAYS

Fig. 4—Compressive strength improved by addition of calcium chloride

II and V cements. The Type II cement gave higher strengths than the Type V cement for each percent of calcium chloride. The highest strengths were obtained with Type II cement and 2 percent calcium chloride. Lowest strengths occurred using Type V cement with no calcium chloride.

DURABILITY TEST RESULTS

These accelerated freezing and thawing tests were conducted on air-entraining concretes when 28 days old. Fig. 5 compares durabilities of concretes cured 1, 3 or 7 days at 50 F prior to freezing in air at 10 F. The concretes subjected to mild cycles of freezing and thawing (17 hours in air at 10 F and 7 hours in water at 50 F) deteriorated slightly faster than the concretes stored at 10 F, especially after only 1 day of curing at 50 F. When no calcium chloride was added the durability of concretes stored at 10 F after only 1-day curing at 50 F was low. For the Type V cement concrete, durability was also low after 3 days of protection.

A comparison of the cements shows that without calcium chloride the concretes containing the Type II cement were the most durable; while with 1 percent calcium chloride there was little difference, and with 2 percent calcium chloride the concretes made with Type V cement were the most

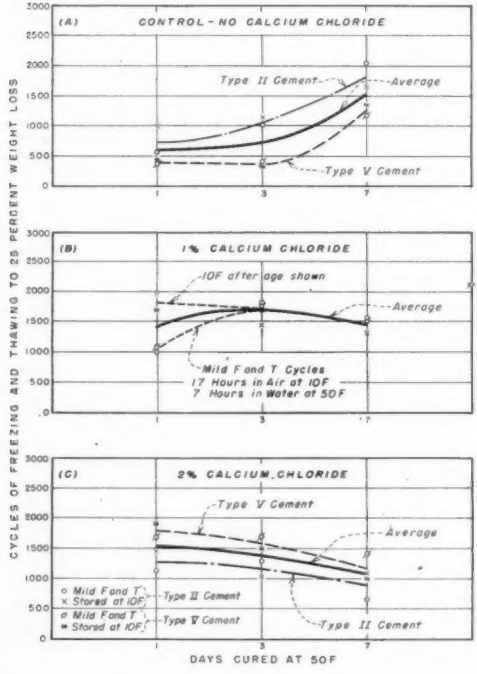
(A) CONTROL - NO CALCIUM CHLORIDE
Type II Cement
Average
Type V Cement
(B) 1% CALCIUM CHLORIDE
10F after age shown
Average
Mild F and T Cycles
17 Hours in Air at 10F
7 Hours in Water at 50F
(C) 2% CALCIUM CHLORIDE
Type V Cement
Average
o Mild F and T
x Stored at 10F
Type II Cement
o Mild F and T
• Stored at 10F
Type V Cement
Type II Cement
CYCLES OF FREEZING AND THAWING TO 25 PERCENT WEIGHT LOSS
DAYS CURED AT 50F

Fig. 5—Durability of air-entraining concrete stored at 10 F or given mild freezing and thawing after initial curing. Concrete tested at 28 days age

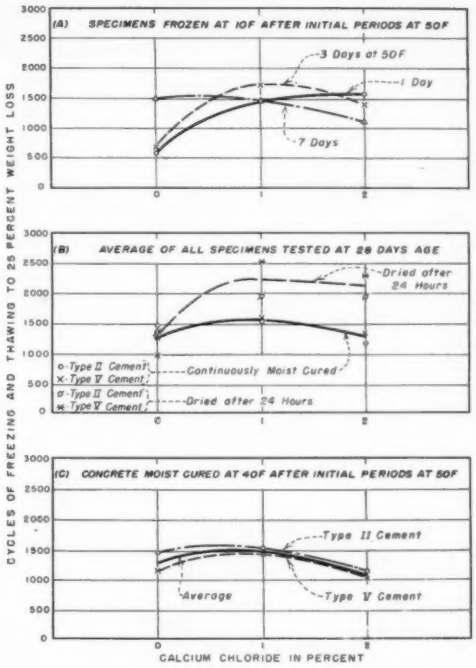
(A) SPECIMENS FROZEN AT 10F AFTER INITIAL PERIODS AT 50F
3 Days at 50F
1 Day
7 Days
(B) AVERAGE OF ALL SPECIMENS TESTED AT 28 DAYS AGE
Dried after 24 Hours
o-Type II Cement
X-Type V Cement
Continuously Moist Cured
Dried after 24 Hours
(C) CONCRETE MOIST CURED AT 40F AFTER INITIAL PERIODS AT 50F
Type II Cement
Average
Type V Cement
CYCLES OF FREEZING AND THAWING TO 25 PERCENT WEIGHT LOSS
CALCIUM CHLORIDE IN PERCENT

Fig. 6—Durability of air-entraining concrete stored under various conditions. Concrete tested at 28 days age

durable. Average durability of the concretes with calcium chloride was above 1400 cycles, which indicates excellent resistance to freezing and thawing.

In general, these tests show that from the durability standpoint only 1 day of curing and protection at 50 F is required if calcium chloride (1 or 2 percent) is added to the mix. Longer curing is required for strength development.

Graph A in Fig. 6 summarizes the mild freezing tests. Each curve represents the average of the two cements. One percent calcium chloride appears to be optimum for these concretes subjected to accelerated freezing and thawing tests when 28 days old. Another series of tests still underway indicates that calcium chloride decreases the durability of older concretes in which the cement is more completely hydrated.

Graph B of Fig. 6 compares the durabilities of air-entraining concretes given different periods of moist curing. Drying after only 24 hours in the molds appears to be beneficial if calcium chloride is added to the mix and even without calcium chloride no harm is done by such drying. Additional curing did take place during the accelerated freezing and thawing tests. A word of caution, however; the results apply for air-entraining concretes only.

Graph C of Fig. 6 shows that all of the concretes cured at 40 F, after preliminary curing at 50 F, gave excellent resistance to freezing and thawing with little difference between cements or calcium chloride contents.

CONCLUSIONS

The following conclusions are based on results of these tests, the findings of other laboratory studies and on observations of concrete structures in the field.

Compressive strength

1. With a Type II cement, freezing for 3 days after 7 days curing at 50 F was not injurious to the concrete; freezing after 3 and 5 days curing reduced the 28-day strength about 7 percent. The addition of 1 or 2 percent $CaCl_2$ more than compensated for the strength loss due to freezing and produced higher strengths at all ages than the control specimens, without $CaCl_2$, continuously cured at 50 F.

2. With a Type V cement, freezing for 3 days reduced the 28-day strength about 13 percent whether the concrete was cured 3, 5 or 7 days prior to freezing. The addition of 2 percent $CaCl_2$ more than compensated for the strength loss due to freezing.

3. Early freezing was not detrimental provided curing was continued after the concrete thawed. Specimens protected for 3 days at 50 F, followed by 25 days of alternate freezing and thawing, then moist curing at 50 F, produced higher strengths after age 60 days than specimens continuously cured at 40 F.

4. There was little difference in the 28-day strengths of specimens stored at 40 and 50 F. The 28-day strengths of specimens containing Types II and V cement, stored at 32 F were 94 and 78 percent of that of the control specimens cured at 50 F.

5. The 28-day strengths of specimens containing Types II and V cement and 2 percent $CaCl_2$ were 18 and 30 percent higher than for specimens without $CaCl_2$.

Durability

1. Air-entraining concrete placed, cured, and protected at 50 F for 1 day gave excellent resistance to freezing and thawing for both cements when 1 percent calcium chloride was added (Fig. 5).

2. Air-entraining concrete, without calcium chloride, required curing and protection for 3 to 7 days depending upon the rate of hydration of the cement (Fig. 5).

3. The effects of age, temperature, curing and calcium chloride on the durability of concrete are all reflected by the amount of hydration of the cement. Immature (only partially hydrated cement) concretes tested at early ages show that optimum durability is obtained when 2 percent calcium chloride is added. Concretes tested at later ages, when the cement is more completely hydrated, show that higher percentages of calcium chloride reduce durability. Therefore, if concrete is likely to receive severe freezing and thawing at early ages calcium chloride should be added; if the concrete can be protected from 3 to 7 days (depending upon the cement) no calcium chloride is required.

SUGGESTED SPECIFICATIONS

The following specification was formulated considering both laboratory tests and expected variations in the field.

When placed in freezing weather, concrete made with a Type II cement shall contain 1 percent calcium chloride and 4 percent entrained air and shall be kept at a temperature not less than 50 F for 3 days and shall be protected from freezing for 3 additional days. With a Type V cement the same percentages of calcium chloride and entrained air shall be used,

and the concrete shall remain at a temperature not less than 50 F for 7 days. Additional precautions to prevent concrete temperature from falling more than 40 F per hour, shall be observed.

ACKNOWLEDGMENTS

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Title No. 47-35

Precasting aids contractor in achieving economy

Precast Concrete Construction in Canada*

By OTTO SAFIR†

SYNOPSIS

Precast concrete construction techniques as applied to a warehouse, parking garage and retaining wall project are described. Beams and columns for the precast concrete frames of a warehouse were cast flat in multiple forms on the ground slab. Cold weather required special precautions in mixing, placing and curing the concrete.

In a parking garage, precast elements were combined with cast-in-place members to form an essentially monolithic structure. A retaining wall 25-ft high is of precast counterforts and 8-in. wall planks. Principal features of the job were the complicated shape of the counterforts and the weight of the completed units.

INTRODUCTION

In striving for economy in construction, and especially in times of material shortages, the engineer should consider the use of precast concrete. Precasting has become an accepted method of construction and new uses and techniques are being developed constantly.

One of the major reasons for the increased popularity of precast concrete is that it reduces formwork costs. A large proportion, broadly speaking a third to a half, of the total cost of a reinforced concrete structure is in the formwork and precasting offers savings in this respect. This applies particularly to structures at considerable height above the ground where the staging is a big problem or to structures with complicated shapes. In precast structures the forms can be re-used frequently and their first cost divided by the number of times used is therefore small.

The designer need not hesitate to employ complicated shapes which he would be reluctant to use on traditional structures, if such shapes are those best adapted to the particular requirements of the structure. This, in some cases, leads to economy in concrete and steel.

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Since precasting operations can be done on the job-site and at ground level, it is easy to assure close supervision of the placing of the concrete with resulting good workmanship and strength characteristics. It is necessary, however, to fully plan the operations before work starts on the site and close liaison between the designing office and the job is a prime requirement.

When designing and casting precast units, particular attention must be paid to dimensional accuracies. Some inaccuracy is unavoidable and the design should permit sufficient tolerances at the junctions to allow for such inaccuracies. To neglect these tolerances may lead to considerable difficulty during erection. Once permissible inaccuracies have been determined, and they should be stipulated on the drawings, the work at the job-site must adhere to them. A frequent check of the forms is necessary and, if required, they must be adjusted.

The erection of the matured precast members is an operation not hitherto encountered in reinforced concrete but, provided equipment with ample lifting capacity is used, no special difficulties arise if careful consideration is given in advance to the various phases of the work and to the special lifting devices and attachments required. The designer must plan for the stress to be expected in the precast unit during handling, especially when the members are cast on the flat. By careful design of the lifting arrangements it is possible to avoid crack formation in the precast unit.

Precast concrete structures can be divided into two broad categories: (1) where the precast elements are assembled completely "dry" to form larger structures, and (2) where the precast elements form part of an essentially monolithic structure by being assembled and connected through grouting and possibly by being attached to in-situ portions. In some cases a combination of these two methods might be used, *i.e.*, some joints in the completed structure are left open whereas others are closed and the structure made continuous at these points.

PRECAST CONSTRUCTION

Single story warehouse

Some indication of the economy achieved with precast concrete may be taken from the following figures for a single story warehouse 540 ft long and about 120 ft wide. The owners required the lowest cost structure for storage purposes; only the structural frames and purlins were of reinforced concrete, the roof and wall sheeting consisted of 2-in. tongue-and-groove boarding. The cost of the complete reinforced frame superstructure was $0.90 per sq ft; the complete building, including foundations, slab on grade, walls and roof was $2.25 per sq ft of covered area.

The roof beams were continuous over two spans of approximately 30 ft each; joints were arranged in the outside spans approximately at the point of contraflexure so as to reduce the length of precast beams to be handled and to make the structure statically determinate and not affected by any uneven settlements. The relative position of the two beams at the joint

was secured by a bolt inserted in a vertical direction. The two tapered columns were connected rigidly to the longer roof beam by two vertical bolts and with this beam form single-legged frames to resist wind forces against the outside walls. The other three columns of one bent were articulated, all connections between columns and foundations transmitting only vertical and horizontal forces, and were formed by short steel dowels projecting upward from the foundations and engaging in pockets on the columns which were grouted in when erection was complete. At all other joints resilient expansion jointing was used as an interlay between contact surfaces.

The roof purlins were also precast and, prior to erection, fitted with nailing strips to receive the roof sheeting. The purlins were connected to the main beams with 14 gage bent steel fittings. All main beams and columns are 10 in. wide and they were cast on the grade slab on the flat in 30-in. deep forms, three on top of one another. They were separated from the slab and from one another by waxed paper. An interval of 24 hours was allowed between casting subsequent layers of beams and columns; every day one complete set for one bay was cast, consisting of four main beams, five columns and 19 purlins. The forms were made of plywood stiffened on the outside and the exact spacing was maintained by yokes above the forms to avoid tie bars penetrating the concrete. Plywood jigs were used when setting up the tapered columns. Since casting was done during cold weather, heated aggregate and water were used for the concrete and the cast members were protected by tarpaulins. Live steam was blown under the protective covers after the initial set had taken place.

Because of the ease of placing, a low water-cement ratio concrete mix was used and the concrete consolidated with internal vibrators supplemented by hand tamping and rodding. The cement content was generally 6 sacks per cu yd of concrete. A sharp, coarse, river sand was used for the fine aggregate. After some trial batches, the originally designed mix was varied and set as 1 sack of cement (87½ lb) to 2½ cu ft of sand and 3 cu ft of gravel not exceeding 1½ in. With a slump of not exceeding 3 in., 7-day compressive strengths of 3500 psi and more were obtained consistently. Similar concrete mixtures were used on the other structures described.

Fig. 1 shows the concrete members laid out in three bay lots on the ground slab just prior to erection. At the far end the recently cast members are maturing under protective covers. Fig. 1 also shows the temporary trestles which held the columns during erection.

Fig. 1—General view of warehouse site prior to erection. Precast members in background are curing under protective covers, erection trestles for columns shown in foreground.

Casting beams and columns on the flat in multiple forms has proved to be very economical. It necessitates, however, special provisions for lifting and requires more work in re-slinging. In this particular case, all slinging was done with specially made shackles attached to the concrete member by bolts through pipe sleeves placed when precasting. This meant two sets of pipe sleeves for the heavy members, the cost of which is an item of some importance.

When a precast member is lifted from the place in which it has been cast, considerable adhesion has to be overcome. For this reason, specially constructed unsymmetrical shackles were used. Fig. 2 shows these special shackles and one of the large roof beams being lifted off the stack. The heaviest precast concrete member weighed almost five tons. Fig. 3 shows the wind bracing consisting of round steel bars with turn-buckles used in the first bay to give stability to the structure against lateral forces. Fig. 4 is an end view of the first half of the building. It shows two temporary gable uprights in position; these were moved later to the end of the second half of the building.

Fig. 2—Lifting beam with eccentric shackles

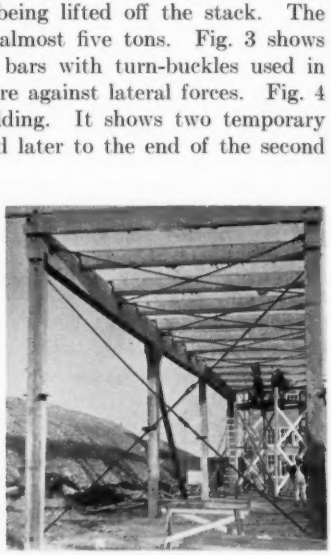

Fig. 3—Wind bracing between first two frames

Fig. 4—End view of first half of warehouse. The two temporary gable uprights on either side of the center support were removed after completion of rest of building; precast beams are being formed in the foreground.

These warehouses, of which two almost identical ones were built, are examples of the first category of structures, *i.e.*, they were assembled from precast concrete members completely "dry."

Parking garage

Precast concrete beams were used in the construction of a two-story garage building (second floor is to be added at a later date). The lot was to be used for parking cars as much as possible while building operations were going on and a cast-in-situ structure was therefore ruled out. The main beams of the building are spaced 26 ft apart; the total width is 132 ft; the free spans between the columns are approximately 38 ft, two pairs of two columns each carry 52 ft long main beams which cantilever to the outside approximately 9 ft and project into the center bay and support the 26 ft long center beams. The main beams, cast on the ground in 52 ft lengths, weigh eleven tons. The main beams carry 26 ft long secondary precast beams weighing 2½ tons each, which are spaced 12 ft center to center. The secondary beams rest in pockets on the main beams. Fig. 5 shows the placing of these beams. These pockets were grouted and the beams were made continuous by placing top steel across the main beams when the 5-in. floor slab connecting the secondary beams was cast in place. All the precast beams projected approximately 2 in. into the slab to take up slight inaccuracies.

Fig. 5—Placing secondary beams in pockets of main beam. Note the pipe sleeves near underside of beams to support formwork for the slab.

The formwork for the slab was assembled in panels and supported on the secondary beams which were cambered to allow for the deflection under the dead weight of the slab. The dead weight of the structure is carried on the secondary beams as beams of single span and rectangular cross section. For the surcharge, these beams act as T-beams which are continuous over several spans. The columns were constructed in-situ.

The main and secondary beams were precast in an upright position on a temporary casting bed constructed of lumber. The beams were cast against one another. Having stripped one side of the forms, a thin dividing sheet was placed against the green concrete (usually 24 hours old). This divider was either hardboard or a thin plywood sheet and the side form was set up again at the correct distance. Lifting eyes were formed by ordinary bent reinforcing bars anchored in the concrete. Because of the importance of

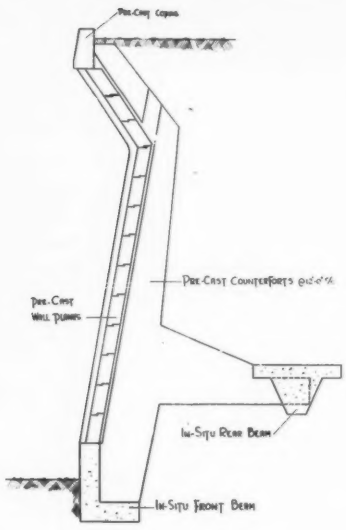

Fig. 6—Section of retaining wall

these lifting eyes, it was thought advisable to use two such bars at each point although one would have been of sufficient strength. When lifting these beams, adhesion was overcome by lifting one end first before attempting to put the whole weight of the beam on the crane. The 52-ft beams required slight additional reinforcement because of this operation.

This structure is essentially monolithic. Only the joints between the cantilever ends and the center portions of the main beams were left open, similar to expansion joints in traditional structures.

Retaining wall

The essential components of the 25 ft high retaining wall shown in Fig. 6 were also produced by precasting on the site. The wall consists of counterforts spaced 12 ft apart supporting in their grooves the 8 in. thick precast wall planks and coping. The front and rear foundation beams of the wall were cast in-situ. The counterforts were precast on the ground in a horizontal position on thin plywood sheets carried on sleepers and sand fill. The counterforts are 18 in. wide and the edge forms were made exactly to this dimension. They could be used once every 24 hours. These counterforts, weighing 12 tons, had to be transported approximately 200 yd on semi-trailers. They were raised by being slung at three points which were positioned so that when raising one edge, the top corner would free itself first without setting up any undue stresses in the tail portion of the counterfort. In erection the counterforts were slung only from the hooks near the top bend (Fig. 7). They were then hanging almost vertically, the front edge touching the foundation first; further lowering would bring the tail into contact with its found-

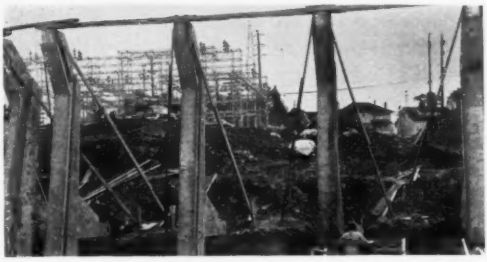

Fig. 7 (Right)—Raising of counterfort

Fig. 8 (Above)—Counterforts in position

ation. Fig. 8 shows the counterforts in position. The two lifting eyes projecting from the upper face of the counterforts precluded the casting of one counterfort on top of the other. The position of the lifting eyes was carefully calculated and checked by small scale model tests. The wall planks were cast in several layers separated by waxed paper, the lowest layer on a sand bed covered with waxed paper. They were lifted by means of a vacuum mat.

This retaining wall is an example of the possibilities given by precast technique for using apparently complicated shapes which are well adapted to the particular purpose of the structure.

The shape of these counterforts was due to the following considerations. The edge of the yard to be retained by this wall had to be right up to the property line beyond which the foundation of the wall could not project. By sloping the counterfort backwards over most of its height, a considerable reduction in the earth pressure results. The rear beam was set at a higher level than the front foundation beam to reduce the amount of excavation and by increasing the length of the heel of the counterfort, the same stability of the wall was achieved as with the rear beam at a lower level closer to the wall. Furthermore, this arrangement reduces the bending moment in the counterfort itself.

A comparison with a traditional reinforced concrete retaining wall showed a steel saving of 50 percent and of concrete of 30 percent. The amount of formwork was small. Against the savings, of course, has to be set the cost of erection, but the over-all result showed a saving in costs of approximately 30 percent.

NEW FIELDS

Precasting technique opens up many new fields for reinforced concrete structures, fields which up to now were the prerogative of structural steel.

A promising field for precast concrete appears to be the construction of larger span buildings. There are, of course, limitations in the size of the precast members, set mainly by the handling equipment. By assembling a multitude of precast elements, larger structures can be formed as has been suggested recently in the design for an arena building giving a free span of 140 ft. In this design, three hinged arches are supported on cantilever ends of three hinged frames formed by the external columns and the cranked balcony girders.

Design of prestressed concrete members imposes new requirements. The author briefly discusses them and their effect on the design adopted for the Walnut Lane Bridge.

Factors in Prestressed Girder Design*

By M. FORNEROD†

SYNOPSIS

Following a general description of the Walnut Lane Bridge the stages of loading the prestressed concrete girders and the sequence of construction operations as they affected the design are discussed. The owner's stress limitations are listed and longitudinal bending in girders, transverse bending in stiffener diaphrams, and shear and principal tensile stress in the girder are considered. Safety factors are considered for various loading conditions.

INTRODUCTION

The Walnut Lane Bridge, Philadelphia, Pa., the first major prestressed concrete bridge in the United States, is already well known to engineers. This design was adopted after comparative estimates showed that a 150-ft solid spandrel stone-faced arch bridge then under consideration would have cost about $300,000 more and also after serious consideration of the esthetic merits of both designs by the city's Art Jury.

The bridge has one central main span of 160 ft‡ and two approach spans of 74 ft each. Each span consists of a series of I-shaped girders with a lubrite plate sliding support at one end and a fixed steel plate hinge at the other. The main span required 13 girders 6 ft 7 in. deep with a cross section as shown in Fig. 1.

Both for esthetic and economic reasons the cross section of the seven girders in each approach span was made identical to that of the main span girders. However, the approach span girders are prestressed with a smaller number of wires because of the reduced bending moment.

A horizontal construction joint separates the prestressed girders from the concrete deck slab which is being placed in-situ after all the girders are prestressed and installed. Girder design, therefore, had to take into consideration the load imposed by the bridge slab. This additional dead load, while

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‡Center to center of piers.

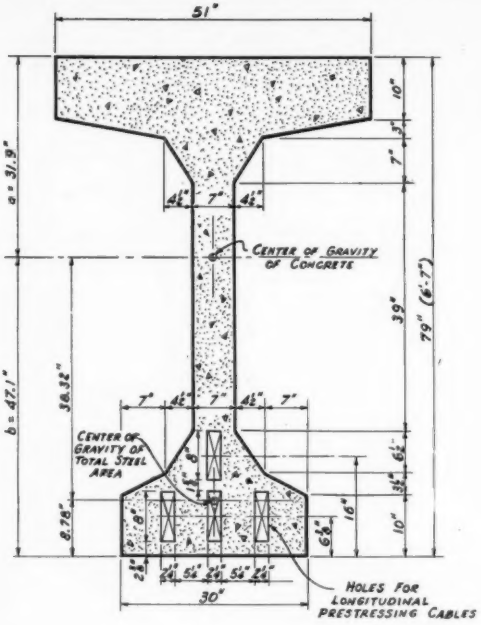


Fig. 1—Girder cross section at mid-span

acting as a live load on the prestressed girder, is nevertheless of permanent nature and can be accurately determined. For this reason the adopted design criteria for this loading should be correspondingly more liberal than for normal live load.

To minimize differential deflections between adjacent girders, nine transverse stiffener diaphragms are placed in the main span and four in each approach span. Transverse prestressing cables at the top and bottom of these stiffener diaphragms insure their action as continuous girders over elastic supports.

Expansion and contraction joints are arranged over each pier across the whole width of the bridge to provide for anticipated movements of the girder ends.

Loading stages and sequence of construction operations

The method of prestressing and the sequence of operation are important in determining intermediate loading stages for which separate design criteria apply.

The girders are cast, cured, prestressed and grouted on a scaffolding so that they can be jacked transversely on specially prepared sliding tracks into their final positions. Prestressing deflects the girder upward sufficiently to lift it off its original cribbing and make it act as a single span girder with all of its dead load transferred to the end supports.

The second loading condition occurs when the girders are in their final position and the roadbed and sidewalk concrete is placed. This permanent live load increases the compressive concrete stresses in the top fiber of the girder and decreases the compressive concrete stresses in the bottom fiber.

The third and final loading condition is the loading of the finished bridge with the working live load. This live load was specified as a standard H-20-44 AASHO load pattern. An impact coefficient of 17 percent was required for the span under consideration. The above load pattern comprises a uniformly distributed load of 640 lb per ft of traffic lane and a moving concentrated load of 18,000 lb for the critical moment and 26,000 lb for the critical shear section.

It is important to note that a "post-stressing" method was used. As described in the literature on prestressing,* the Magnel-Blaton system uses cables the wires of which, after installation in longitudinal holes in the cured girder, are stretched in pairs and anchored at the girder ends with so-called "sandwich" plates and anchor plates before release of the jacking force. The girder end blocks must so distribute the high local stresses under the anchor plates that no critical stress condition occurs in the normal girder section.

After prestressing is completed the longitudinal holes in the girders are grouted to protect the wire from corrosion and to establish bond between the wire and the girder concrete. Tests have shown that properly placed grout considerably improves the crack distribution in a girder.

Post-stressing considerably reduces the losses in the prestressing wire due to shrinkage of the girder. Much of the normal shrinkage takes place before prestressing is started. Nevertheless, a liberal allowance was made in the design for wire stress losses due to continued shrinkage as well as to plastic flow in the concrete and creep in the steel wire. A total allowance of 13 percent of the initial wire stress was made for their combined effect. This is in accordance with recommendations of Magnel† whose experiments and tests give a reliable indication of these stress losses under various load conditions.

To clearly differentiate between stress conditions before and after stress losses, the stage immediately after prestressing is designated as the "initial" phase whereas the phase when all stress losses are assumed to have taken place is called the "ultimate" phase.

Materials

The owner's specifications required a minimum 28-day concrete cylinder strength of 5400 psi. The same minimum concrete strength was required at the age of prestressing. The concrete furnished exceeded this requirement by 20 to 25 percent.

At the time the bridge was designed it was still uncertain whether American manufacturers would be able to furnish, in time, a suitable type of wire for

*Thomas, Frederick G., "Prestressed Concrete," Appendix II, Bibliography, *Proceedings*, Institution of Civil Engineers, London, Feb. 1949.
†Magnel, Gustave, *Prestressed Concrete*, Concrete Publications Ltd., London, pp. 133-165.

prestressing. The wire specifications conformed to the minimum requirement for Belgian wire, namely, a minimum yield strength (0.2 percent offset) of 160,000 psi and a minimum ultimate strength of 210,000 psi.

However, after some development work, the John A. Roebling's Sons Co. furnished a suitable wire whose characteristics considerably exceeded the minima required by the owner. Fig. 2 shows that its 0.2 percent yield strength was approximately 215,000 psi and the ultimate strength was 247,000 psi. When it became known that this better wire would be available all drawings had been approved, the contract was let, and no time was available for radical changes in design. The Roebling wire was therefore installed in preassembled cables of the same cross section and stressed to the same initial stress as originally planned for the Belgian type wire.

The change in wire quality raised the cracking limit of the girder slightly and increased the failure resistance of the girder to a point where the crushing strength of the concrete became the governing factor rather than the breaking

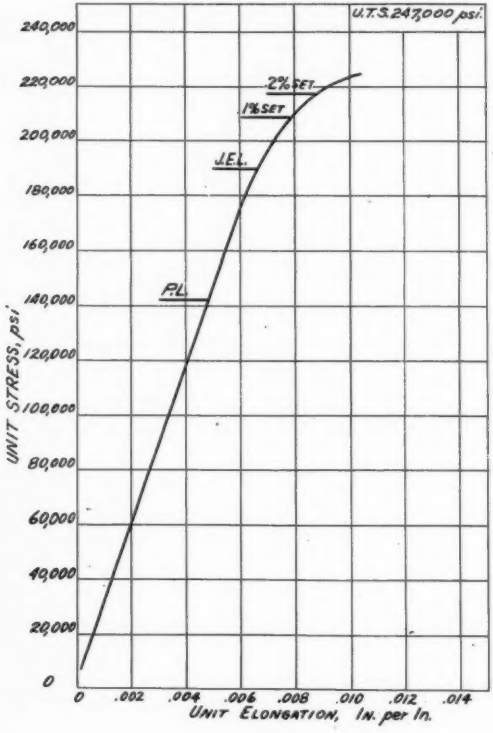


Fig. 2—Stress-strain curve of 0.276-in. prestressing wire

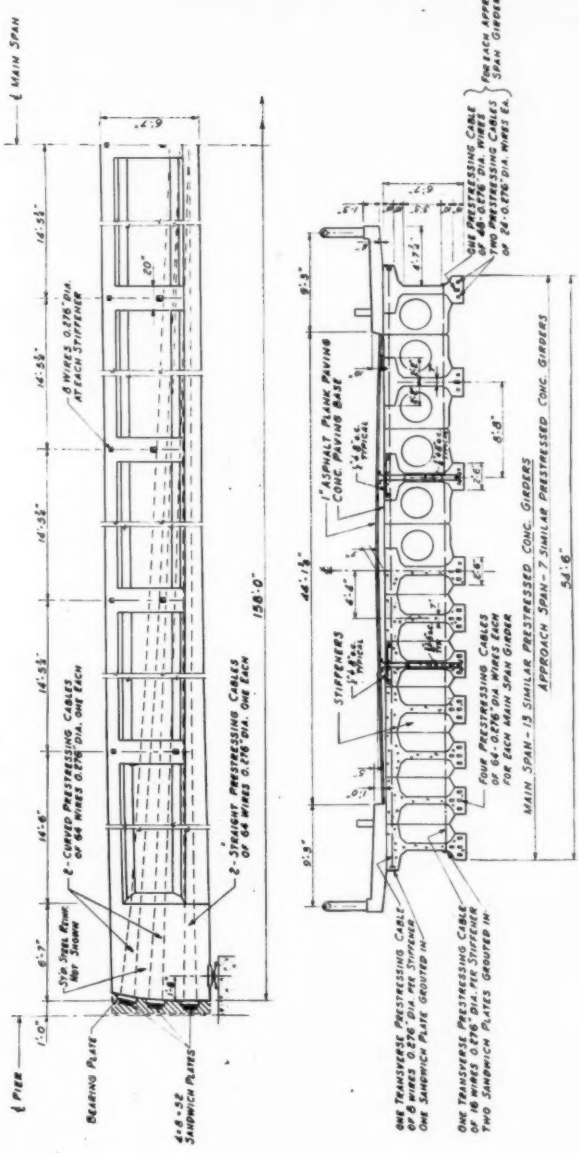

℄ MAIN SPAN
℄ PIER
BEARING PLATE
4-1·0 × 1·2 SANDWICH PLATES
2 - CURVED PRESTRESSING CABLES OF 64 WIRES 0.276" DIA. ONE EACH
2 - STRAIGHT PRESTRESSING CABLES OF 64 WIRES 0.276" DIA. ONE EACH
8 WIRES 0.276" DIA AT EACH STIFFENER
STIFFENERS
1" ASPHALT PLANK PAVING
CONC. PAVING BASE
ONE TRANSVERSE PRESTRESSING CABLE OF 8 WIRES 0.276" DIA. PER STIFFENER ONE SANDWICH PLATE GROUTED IN
ONE TRANSVERSE PRESTRESSING CABLE OF 16 WIRES 0.276" DIA. PER STIFFENER TWO SANDWICH PLATES GROUTED IN
FOUR PRESTRESSING CABLES OF 64-0.276" DIA WIRES EACH FOR EACH MAIN SPAN GIRDER
MAIN SPAN - 13 SIMILAR PRESTRESSED CONC. GIRDERS
APPROACH SPAN - 7 SIMILAR PRESTRESSED CONC. GIRDERS
ONE PRESTRESSING CABLE OF 48-0.276" DIA. WIRES TWO PRESTRESSING CABLES OF 24-0.276" DIA. WIRES EA.
FOR EACH APPROACH SPAN GIRDER
158'-0"
54'-6"

Fig. 3—Typical elevation and cross section of bridge

strength of the wires. This is borne out by the type of failure in the 160-ft full-sized girder tested to destruction at the bridge site.*

DESIGN

The design of the prestressed girders consisted of finding an economical and practical girder cross section, determining the number of prestressing cables, size of end anchorage blocks and of stiffener diaphragms to satisfy the owner's stress limitations. Fig. 3 indicates the dimensions and reinforcing details for the main span girder which is 154 ft 8 in. from center to center of bearing.

Owner's stress limitations

Conventional working stress limitations adopted were:

	Prestressing wire		
Maximum initial stress	126,000 psi	Maximum working stress after losses due to shrinkage and plastic flow	110,000 psi
	Concrete		
Maximum initial compressive stress (dead load only)	2150 psi	Initial tensile stress in top fiber (for dead load only); no tension allowed	0 psi
Maximum compressive working stress after losses due to shrinkage and plastic flow (for total load)	1900 psi	Minimum residual compressive stress in bottom fiber under total load after all losses due to shrinkage and plastic flow	50 psi
Maximum principal tensile stress	215 psi	Maximum initial local compressive stress under end anchor plates	3000 psi

These limitations were primarily to assure a minimum safety against cracking of the girders since, up to the cracking limit, the girder behaves nearly like an elastic structural member of homogeneous material. Beyond the cracking limit such maximum stress limitations do not represent a reliable measure of safety, particularly in the region near failure. Near the failure load the prestressed girders are in a semi-elastic state in which the shape of the cross section and the stress-strain characteristics of the materials considerably affect the bending resistance. The maximum compressive stresses for concrete as specified are therefore no measure for ultimate failure load, but they do prevent the designer from overstressing the girders; an undesirable practice. Over-prestressed girders are brittle and fail rather suddenly since the cracking limit is close to the actual failure load.

Longitudinal bending

Table 1 shows calculated stresses for successive loading conditions for both the initial conditions before any wire stress losses and for the ultimate condition, a stage when it is assumed that all wire stress losses due to shrinkage, plastic flow, and creep of steel amounting to 13 percent have occurred. Table

*Magnel, Gustave, "Prototype Prestressed Beam Justifies Walnut Lane Bridge Design," ACI JOURNAL, Dec. 1950, *Proc.* V. 47, p. 301.

TABLE 1—CALCULATED STRESSES AT MIDSPAN DUE TO BENDING IN TYPICAL MAINSPAN GIRDER* OF WALNUT LANE BRIDGE

Loading condition	Load increment, lb per ft	Total load, lb per ft	Midspan bending moment due to total load, ft-kips	Concrete stresses				Wire stresses		Section properties				
				Initial		Ultimate		Initial	Ultimate	Area, sq in.	Center of gravity from bottom, in.	Moment of inertia, in.⁴	Section modulus, in.³	
				Top, psi	Bottom, psi	Top, psi	Bottom, psi	$n = 6$	$n = 6$				Top	Bottom
(a) Prestressing plus dead load, girder	1800	1800	5377	+1165	+1800	+1260	+1200	−126,000	−110,000	1347	47.1	1,085,000	34,000	23,050
Dead load road deck and miscellaneous dead load	300		898	+ 317	− 467	+ 317	− 467	− 2800	− 2800					
(b) Prestressing plus dead load girder plus road deck and miscellaneous		2100	6275	+1482	+ 1333	+1577	+ 733	−128,800	−112,800					
Live load and impact	445		1330	+ 448	− 543	+ 448	− 543	− 3200	− 3200	1495	43.3	1,273,000	35,700	29,400
(c) Total working load		2545	7605	+1930	+ 790	+2025	+ 190	−132,000	−116,000					

*Span center to center of bearings—154 ft 8 in.

1 also gives the section properties of the main span girder before the prestressing cables are grouted in and after grouting when the values are calculated on the basis of a transformed section.

Fig. 4 shows typical stress diagrams for the loading conditions of Table 1. The stresses due solely to prestressing are of academic interest only as they can never occur by themselves. As soon as the prestressing becomes effective the girder deflects upward off its cribbing, bringing into action the dead load of the girder simultaneously with the prestressing. This condition is shown under (a) in Table 1. It can be noted that the stresses are exceptionally conservative and that this is not a critical condition.

The next important loading stage is after the girders are installed and assume the additional dead load of the road deck concrete and other miscellaneous dead loads. This loading stage (b) (Table 1 and Fig. 4) produces neither tensile stresses nor critical compressive stresses even in the ultimate condition.

It is in the final loading stage under full working load, (total dead load plus specified live load) that the compressive stresses in the top fiber of the girder approach a limiting value (condition (c) in Table 1 and Fig. 4).

The bottom fiber is not subjected to any tensile stress and the residual compressive stress at the bottom is in excess of the minimum of 50 psi required by the specifications. As previously indicated, these stresses are no direct criterion for the safety against failure but they do control the cracking limit of the girder as will be noted from Table 2, which shows the safety factors against cracking.

Transverse bending and stiffening diaphragms

Transverse stiffener diaphragms are arranged at regular intervals to minimize differential deflections of the girder. Such stiffeners become more effective as their bending stiffness increases. For this reason they are also prestressed at top and bottom by transverse cables (Fig. 3). The top cable of the main span stiffener diaphragm consists of eight wires while the bottom cable contains 16 wires. The transverse cables are prestressed in the same manner as the longitudinal cables, after all girders are assembled and the butt joints of stiffener beams are grouted. The bending in the stiffener beams can be calculated approximately by applying the theory of a beam on elastic supports. Approximate load distributing curves are drawn to fit in between the two rectangles for the limiting cases of a stiffener beam of infinite stiffness and one of zero stiffness. Such curves must be drawn for the simultaneous loading of a stiffener with two or three traffic lanes to determine the most critical load position. The moment in the transverse stiffener beams can then be calculated as the difference between external load moments and the moments created by the approximated girder reactions. These calculations, while very approximate, show that under the most severe loading conditions the stresses due to prestressing in the stiffeners are sufficient to offset bending stresses due to deflection of longitudinal girders.

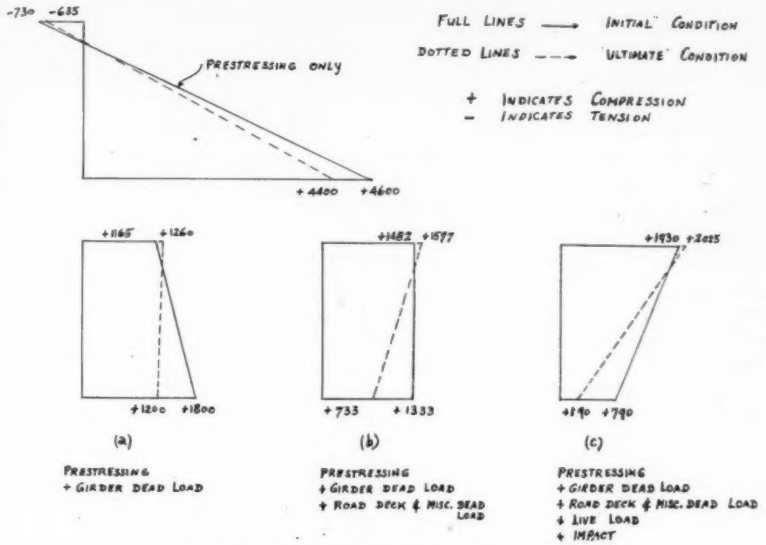


Fig. 4—Typical stress diagrams for mid-span section

Shear and principal tensile stresses near ends of girder

Design of end anchorage blocks in beams prestressed by the Magnel-Blaton system requires special and careful consideration. While the grout injected into the cable interstices effectively bonds the cable wires to the concrete of the girder, it is assumed in the design that the end anchorage plates are relied upon to transfer the total force in each cable to the concrete. The total initial force in one of the main span cables is approximately 240 tons and the maximum initial concrete stress under the bearing plates is in the neighborhood of 3000 psi. The end blocks must therefore distribute these local horizontal stresses over the end section of the girder, a portion of which is also critically stressed by maximum shear forces. In these transition sections between the girder ends and the normal girder section the horizontal and vertical stresses act simultaneously together with a local bending moment the magnitude of which is not exactly known. An assumption has been proposed by Magnel who assumes a parabola of the third degree as the stress distribution created by this local moment.*

Using this method the maximum stresses in two planes at a selected critical point in the end blocks were determined. A Mohr's circle then gave the principal stresses at that point. For the main span girder end blocks, the principal tensile stress is 110 psi and the principal compressive stress is 545 psi. These values are sufficiently conservative to assure freedom from cracks at the working load, as the results of the test girder loadings demonstrate.

*Magnel, Gustave, "Design of the Ends of Prestressed Concrete Beams," *Concrete and Constructional Engineering*, London, May 1949.

Design criteria and safety factors

Before construction was started studies were made to forecast as closely as possible the degree of safety of the girders; first against cracking and second against failure. The criteria governing these conditions obviously differ considerably. Safety against cracking can be calculated on the basis of the elastic behavior of a homogeneous section considering influences of prestressing, dead load and external loading to find a resisting bending moment at which the modulus of rupture of the concrete would be reached. To determine the modulus of rupture the recommendation of Douglas E. Parsons* was used. In Table 2, pertinent values for determination of the cracking limits are shown for three cases.

(a) For the cases in which any and all bending resistance of concrete is discounted as in block construction or in case vertical shrinkage cracks are assumed to exist before prestressing begins.

(b) For monolithic construction with the assumption of an uncracked concrete resisting bending to the full extent of its modulus of rupture as calculated by the Parsons formula.

(c) Values obtained from actual loading test on 160-ft girder at the bridge site. (In this case a vertical crack was observed before stressing operations were begun.)

It is seen that in the worst case cracking does not occur until the girder has been loaded with 2½ times the full live load.

*Parsons, D. E., Disc. "Prestressed Concrete Design Principles and Reinforcing Units," ACI JOURNAL, Nov. Suppl., 1943, *Proc.* V. 39, p. 528-2.

TABLE 2—CRACKING LIMIT—WALNUT LANE BRIDGE GIRDERS

	(a) Modulus of rupture of concrete assumed equal to zero —block construction or vertical shrinkage cracks	(b) Monolithic construction. Concrete assumed to be uncracked	(c) Walnut Lane test girder result
Section modulus bottom fiber, in.³	23,050	23,050	
Concrete prestress ultimate — bottom fiber, psi	4400	4400	
Modulus of rupture in bending of concrete,* psi	0	740	
Crack resisting stress, psi	4400	5140	
Crack resisting moment, ft-kips	8450	9860	9555†
Cracking safety factor based on total load $\frac{M_{crack}}{M_{total}} = \frac{M_{crack}}{7605}$	1.1	1.3	1.25
Cracking safety factor based on live load only $\frac{M_{crack} - M_{DL}}{M_{LL}} = \frac{M_{crack} - 6275}{1330}$	1.6	2.7	2.5

*Parsons, D. E., Disc. "Prestressed Concrete Design Principles and Reinforcing Units," ACI JOURNAL, Nov. Suppl., 1943, *Proc.* V. 39, p. 528-2. $f_{e\ bending} = 1.8 \times (f'_c)^{\frac{1}{2}}$. For $f'_c = 5400$ psi, $f_{e_b} = 740$ psi.

†A vertical shrinkage crack developed before prestressing due to unusual exposure of girder.

To predict the approximate safety factor against failure the semi-elastic behavior of the girders near the failure load must be considered and it seems reasonable to assume a rectangular distribution of longitudinal compressive stresses near the top of the girder. The total force represented by the compression zone must be in equilibrium with the total force in the wires before breakage.

As explained earlier it was anticipated that the strength of the concrete rather than the strength of the wire would govern the actual failure of the girders. Calculations based on the above assumptions were made for the minimum anticipated concrete strength in the installed girders resulting in a midspan failure moment of approximately 20,000,000 ft-lb.

If the safety factor is based on the ratio of maximum midspan bending moment at failure to total working load moment, its value is approximately 2.5. If the excess of failure load moment over total dead load moment is compared to the live load moment, the factor of safety is approximately 10. The high value in this particular design is due to an exceptionally high ratio of dead load to working load of 0.82.

The results from the girder tested to failure at the bridge site corroborated these calculated safety factor values.*

CONCLUSIONS

With the exception of span length and external load, the most important factors affecting the design of a prestressed girder are the proportion of dead load to total load, shape of cross section, method of prestressing and of erection, and the total allowance made in the design for steel stress losses. The latter depends on the tolerances specified for material qualities and for dimensions and on the accuracy, consistency and uniformity of initial prestress in the prestressing wires.

Design criteria for a prestressed structural member should logically be made for various loading stages depending on whether they are permanent and well-defined or temporary. The most important requirements are those for minimum safety against both cracking and failure. Safety against cracking is influenced mainly by the ratio of initial concrete compression stress due to prestressing at the bottom fiber to the stress differential at the bottom fiber due to working load. The higher this ratio, the higher is the cracking limit. Up to the cracking limit the girder acts as a fully elastic structural member.

The second and most important safety requirement, that against failure, is for a given ultimate wire strength dependent upon the ratio of steel percentage to actual crushing strength of concrete. This ratio sets a limit to safety against failure inasmuch as, if this ratio is small, failure will occur due to breaking of the wires and the safety factor is virtually unaffected by the amount of prestress given the wires.

*Magnel, Gustave, "Prototype Prestressed Beam Justifies Walnut Lane Bridge Design," ACI JOURNAL, Dec. 1950, *Proc.* V. 47, p. 301.

If, however, this ratio is high, failure will occur due to crushing of the concrete in the top fiber. In this case, the failure load is affected somewhat by the amount of prestressing, but this influence is relatively unimportant as compared to the limiting influence of the crushing strength of concrete. It can therefore be said that for all practical purposes the failure load is independent of the cracking limit.

Considerations such as the above are sufficient to make a safe and adequate design. There is, however, ample room for testing and research, particularly with regard to the semi-elastic behavior of prestressed structural members near failure load and with regard to the performance of composite prestressed members and the influence of non-prestressed portions of a structure.

ORGANIZATION

The Henry W. Horst Co., Philadelphia, is the general contractor. The Preload Corp., New York, is sub-contractor for the construction of the prestressed girders. The work is directed by the Bureau of Engineering, Surveys and Zoning of the City of Philadelphia, of which A. Zane Hoffman is Chief Engineer, Samuel A. Baxter, Assistant Chief Engineer, and J. Grandinetti, Acting Principal Assistant Engineer, succeeding E. R. Schofield, now retired.

Discussion of a paper by M. Fornerod:

Factors in Prestressed Girder Design*

By CHARLES C. ZOLLMAN and AUTHOR

By CHARLES C. ZOLLMAN†

The engineering profession is indebted to all those who have brought prestressed concrete back to America. Even more so to the engineers who in articles and papers have been willing to inform American engineers about the experience acquired both in the design and in the field. The author's paper is, therefore, a most valuable addition to our technical literature.

However, this writer is of the opinion that a number of points brought up in the paper require more amplification and he further believes that a number of other important points have been omitted.

CONCRETE BEARING STRESSES UNDER WIRE UNIT PLATES

Among the former, concrete stresses under the wire unit bearing plates require closer examination. It is stated in the paper that a maximum initial concrete bearing stress of 3000 psi has been used under the wire bearing plates although the ACI Code recommends as allowable concrete bearing stress the substantially lower value of $0.25\ f'_c$ or 0.25×5400 or 1350 psi. Therefore, it is of great interest to note that for all prestressed concrete work performed in Belgium, the Belgian Code allows a concrete bearing stress of $1.75\ f_c$ with f_c around 2000 psi, which is quite normal in prestressed concrete work. The allowable concrete stress in bearing would then be 1.75×2000 or 3500 psi. The hundreds, if not thousands, of girders, where such bearing stresses have been used, are still standing up and certainly will do so for a long time to come.

How can this great discrepancy in allowable bearing stresses be justified? For the following reasons this writer believes that the ACI limitations do not apply to concrete stresses under wire unit bearing plates.

1. The concrete area which is to receive the wire unit bearing plates is smoothed off, so that a uniform load distribution is assured at all times.
2. There is no possibility for any concentrated loads under these plates. In bridge seats, the angular rotation at the supports of the girder due to deflection may well—and often does—cause high locally concentrated loads.
3. It is reasonable to assume that the loads are distributed within areas lying between 45° lines drawn from the edges of the bearing plates and the outside faces of the end blocks. Such areas increase very rapidly along these 45° lines away from the bearing plates, thus decreasing the unit concrete bearing stress very rapidly.

*ACI JOURNAL, Feb. 1951, *Proc.* V. 47, p. 469. Disc. 47-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

†Chief Engineer, Vacuum Concrete, Inc., Philadelphia, Pa.

This writer believes that these are *some* of the reasons why no sign of either cracking or crushing in or near the end blocks occurred during the testing to failure of the Walnut Lane test girder.

DESIGN OF WIRE UNIT BEARING PLATES

There is no indication in the author's paper about the design of the wire unit bearing plates; even though these plates not only fulfill an important function, but their cost is a substantial item. Presumably the design is based on the customary flexure formula $f = Mc/I$; the concentrated prestressing forces being in one direction and the concrete reactions, uniformly distributed over the entire plate area, being in the other direction. Designed in this manner and probably based on a working stress of about 30,000 psi for an ultimate tensile strength of the plate of about 75,000 psi, the thickness of $3\frac{1}{2}$ inches was determined for the 64-wire unit. The over-all dimensions of the plate being $12\frac{1}{2} \times 15\frac{3}{4}$ in. the weight of the plate would be about 150 lb. Both thickness and weight seem to be very high—to say nothing about costs—and must be reduced if prestressed concrete is to be made more economical. A first approach to that goal is to question whether or not the flexure formula as such should be used without reservation in the case of plates bearing on their entire surface on concrete and receiving concentrated loads. There are indications that other design assumptions might have to be made and tests carried out recently under this writer's supervision bear this out. Bearing plates designed in accordance with the flexure formula carried without failure a much greater load then they were supposed to according to the computations. A possible explanation might be that the plates have —and unnecessarily so—a factor of safety much greater than have the wires. In other words, the wires reach their ultimate tensile strength and break long before the bearing plates reach their ultimate strength. Why should working stresses not be raised then, particularly as it is remembered that the computations assume a reduction of about 15 percent in the initial prestressing forces and that the variation of these forces due to the live load is only about 4 percent. The stresses in the wires, and thus the forces on the bearing plates, remain fairly constant throughout the life of the member once the assumed losses have—or have not—occurred. Appropriate design assumptions and higher working stresses would materially decrease the weight and the cost of these plates.

This writer has expanded on this seemingly unimportant detail of the prestressing portion of the project because time and again he has found that these plates represent a substantial cost item. This writer believes that the engineering profession will welcome any sound approach to the design of bearing plates which would reduce their size and cost.

DESIGN OF END BLOCKS

The large forces transmitted to the concrete by the bearing plates in the main span girders of the Walnut Lane Bridge are around 1,500,000 lb. This

brings out immediately the importance of the concrete end blocks which in turn have to transmit these forces to the I-shaped section of the girder away from the end blocks. The author brings out carefully, and with emphasis, the special care the design of the end blocks requires so that the amount of ordinary reinforcing bars can be determined. He, however, fails to include any computations on this phase of the general design of the project. The analysis of the end blocks is quite unusual and is not only required for the Magnel-Blaton system, as the author seems to imply, but for all systems, particularly to those which call for few but large concentrated loads. To this writer's knowledge such an analysis hasn't as yet been presented in any American paper. It is hoped that the author will present the analysis of the end blocks of the Walnut Lane Bridge.

TRANSVERSE PRESTRESSING

The question of transverse bending and stiffening diaphragm—transverse prestressing—which the author presents, is of the greatest importance and might well be, in this writer's opinion, one of the most outstanding features of prestressed concrete bridges.

In both structural steel and reinforced concrete design, the distribution of wheel loads has been for many years a highly controversial subject. Tests are still being carried out in laboratory and field to determine whether the distributions assumed really occur. It is customary for designers to accept, without too much questioning and investigations, the distribution factors recommended by AASHO Specifications. However, transverse prestressing alters the picture completely. If properly applied, it is to be expected that all the girders and the carrying slab will work together as one unit, much more so than the standard concrete deck poured on steel girders; whether or not composite action is assumed. If so, what is to be expected concerning the deflections of such a structure. To this writer's knowledge, nowhere in the technical literature, either here or abroad, has the question—and importance—of transverse prestressing been explored. The Walnut Lane Bridge offers an excellent opportunity to explore this problem and again it is hoped that the author will seize the opportunity and do so.

DEFLECTIONS

The important matter of the deflections themselves is, surprisingly enough, hardly broached in the author's paper. Longitudinal prestressing alters the usual picture of deflections, so important for one who wants to "see" how the girder "acts" when it is prestressed, and when it is loaded. Based on previous experience, this writer believes that the deflections will not only be well within the allowable limits specified by AASHO, but that they also will be much smaller than those of a plate girder of the same span. No doubt the figures the author has for this girder, which has a depth of only 1/23 of the span, will bear out this writer's statement. The data on the computed deflections and interpretation thereof will be of great interest.

The important conclusion to be drawn is that shallow prestressed concrete beams deflect little, well within allowable limits, another remarkable feature borne out by the Walnut Lane bridge.

CONCLUSION

For reasons unknown to this writer, the design of prestressed concrete seems to be surrounded with mystery. However, from his own experience, this writer can state and he wishes to emphasize that the design and analysis of prestressed concrete structures do not offer any difficulties—far from it. He therefore is quite disappointed to see that the author in his paper has not chosen to go into details of analysis which would have clarified many questions engineers have, but has rather preferred to make very general statements of which quite a number duplicate those already given in Magnel's and in other papers.

From his daily work this writer can conclude that prestressed concrete is still in the stage where designers require advice and guidance from those who have had experience in this relatively new field. If prestressed concrete is to become more general some of the unwarranted mystery must be removed. It is hoped that the author—or the designer of the Walnut Lane Bridge, as nowhere is it stated who the designer is—will see fit to present some details of the analysis made for this first large prestressed bridge in the United States, a milestone in concrete construction in America.

AUTHOR'S CLOSURE

In his discussion Mr. Zollman raises a number of points which certainly deserve full consideration in any prestressed concrete girder design.

In the girder design under discussion all of these points were fully investigated and the results approved by the Owner's Engineers before construction began. Most of these calculations were referrred to but were not included in complete detail as the paper would otherwise necessarily have become extremely lengthy.

The author agrees that a new approach for the design of wire unit bearing plates is indicated. Since both the concrete and the bearing plates are elastic it would be logical to use load distribution curves arrived at by the theory of elastic foundations on semielastic material.

Further tests on such bearing plates should be made to corroborate these design procedures.

With reference to the design of the end blocks, we can furnish to those interested some ten pages of detailed calculations based on the concepts outlined in the paper. Other examples of such calculations can be found in Professor Magnel's book on prestressed concrete. The author did not include this material in his paper as he hopes to be able to modify and considerably simplify this design procedure in the course of further test and construction experience.

Mr. Zollman's remarks on transverse prestressing emphasize the extra benefit in load distribution derived from transverse prestressing. The author believes that prestressing in two directions makes the use of the theory of beams on elastic supports for the calculation of load distribution applicable for all loads up to the cracking load. Upon the author's suggestion such calculations were made for the elastic girder grid. It would be interesting indeed to be able to obtain actual field measurements of deflections and strains under various loading conditions in the finished bridge. The deck slab undoubtedly participates to some extent in this load distributing function. Although the author prepared the testing program for the test girder, he is not in a position to initiate or authorize such additional field tests as Mr. Zollman's suggestions imply.

It is well known that total deflections very seldom become a limiting factor in prestressed girder design. Only in exceptionally long spans and with very shallow girders must their value be ascertained to be within allowable limits. In prestressed concrete structures total deflections are no longer proportional to the load as dead load deflections are cancelled in the process of prestressing and because usually a negative upward deflection remains. Deflection increments, however, are proportional to the corresponding load increments up to the cracking load.

This is corroborated by the strain and deflection curves (Fig. A) and Table A taken from a paper by the author on the Walnut Lane Girder Tests.*

It is noted that for a load increment of 445 lb per ft the observed deflection increments at midspan are 0.74 and 0.80 in., respectively, averaging 0.77 in.

The calculated deflection increment obtains for $I = 1{,}085{,}000$ in.^{4}, $E_c = 6{,}700{,}000$ psi, $L = 1848$ in., $\Delta M = 16{,}000{,}000$ in.-lb as follows:

$$\Delta\delta = \frac{5}{48} \cdot \frac{\Delta M \times L^2}{E_c \times I} = 0.70 \text{ in.}$$

A comparison of the two values for $\Delta\delta$ indicates that the value of E_c of 6,700,000 psi used for interpretation was somewhat too high.

From Table A it can be seen that a point of zero deflection is reached at a load corresponding to 1.64 times live load and impact. This shows that total deflections are well within allowable limits.

In conclusion the author wishes to point out that his paper was intended to cover the essentials of the design only. He fails to see the necessity to publish detailed calculations especially since all elementary design procedures in prestressed concrete have been amply described in many textbooks and a very extensive literature. The author does not share Mr. Zollman's opinion that the design of prestressed concrete structures is surrounded with mystery. Most engineers agree that the basic principles are very simple. Such questions which today still need clarification are generally in the realm of development work and research, a fact amply demonstrated by the papers presented at the recent MIT conference on prestressed concrete.

*Fornerod, M., "Load and Destruction Test of 160-Ft Girder Designed For First Prestressed Concrete Bridge in U.S.A.," V. X, *Publications of The International Assn. For Bridge and Structural Engineering*, Zurich, 1950.

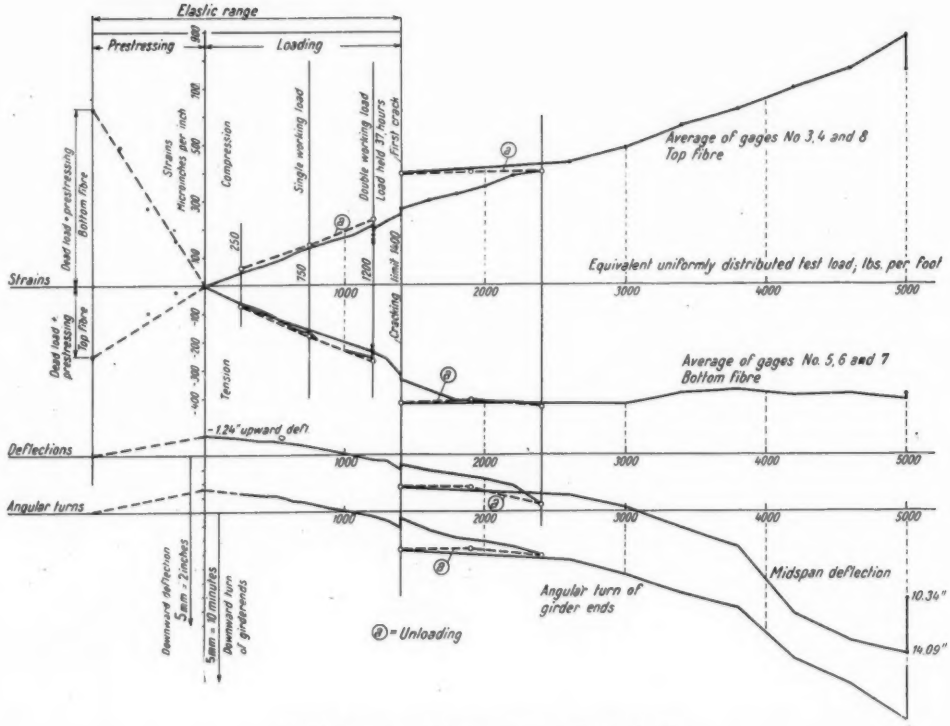


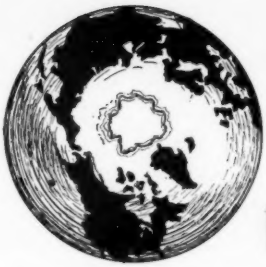
Fig. 4A—Strain and deflection curves for test load

TABLE A—STRESS TABULATION FOR WALNUT LANE TEST GIRDER

TABLE A—STRESS TABULATION FOR WALNUT LANE TEST GIRDER

Loading condition	Load increment	Equivalent uniform superimposed testing load lbs./ft.	Total Equivalent uniform load lbs./ft.	Total moment foot kips	Concrete stresses at midspan				Measured deflection inches midspan − upward + downward
					Calculated (with 10% loss at age of test)		From strain measurements eav. = 6700000 psi		
					Top	Bott.	Top	Bott.	
Due to prestressing (at age of 30 days)					− 655	+4150	—	—	
Dead load Girder (incl. grout)	1800	0	1800	5,377	+1240	+1350	+ 867[1] +1240	+2200[1] +1248	− 1,29″
D.L. Girder + superimposed D.L. of road-deck and misc. D.L.	250 50	250 300	2050 2100	6,125 6,275	+1492 +1542	+1034 + 973	+1562[2] —	+ 852[2] —	− 1,24 —
Above D.L. + live load + impact	445	745	2545	7,605	+1990	+ 430	+2182	+ 110	− 0,50″
Above D.L. + double live load + double impact	445	1190	2990	8,935	+2435	− 112	+2562	− 358	+ 0,30
Load at which first crack opened	210	1400	3200	9,555	+2642	− 362	+2994	− 888	+ 1,00
Shortly before failure	3600	5000	6800	—	—	—	—	—	+14,1″
At failure	200	± 5,200	± 7000	20,800 ± [3]	—	—	—	—	± 25″

Remarks. [1]) For E_c = 3,500000 psi measurements unreliable (see text).
[2]) Extrapolated value for 250 lb. load plus calculated value for dead load.
[3]) Produced by approx. $38^1/_2$ tons on jacks 2, 3, 4, 5, 6, 7 and 60 tons of ingots at midspan.



Letters from Readers*

BY WAY OF SYNOPSIS

JOSEPH P. RYAN presents information on the conditions which prevail for the corrosion of steel reinforcement in concrete.

BOYD G. ANDERSON and EDWARD COHEN offer further evidence in favor of the plastic theory of design in answer to a November "letter."

Relationship of Fly Ash and Corrosion (LR 47-60)†

A reader in the September JOURNAL raises the question: "Does fly ash in concrete increase the corrosion of steel reinforcement?" He draws particular attention to the possibility that the presence of sulfur compounds in the fly ash may be highly objectionable.

Recently the Sanitary District of Chicago, along with a number of other interested public bodies and several industrial concerns, investigated the use of fly ash as an ingredient in concrete to determine particularly its effect on early and later strengths, and its effect upon the resistance of concrete to freezing and thawing and to sulfate attack. The latter effect is of particular interest to those who build sewers, sewage pumping stations and sewage treatment works.

The question the reader asks has never been investigated as such by us, but the accompanying theoretical discussion of the conditions which prevail for corrosion of steel imbedded in concrete made either with or without fly ash may be of interest.

Corrosion of steel reinforcement

Reference to the environment of steel, especially reinforcing steel, in promoting or otherwise inhibiting corrosion, may be found in the *Corrosion Handbook* edited by H. H. Uhlig and in *Metallic Corrosion, Passivity and Protection* by Evans.

Uhlig[1] says: "The effect of the hydrogen ion activity (pH) of water on the corrosion of iron at 72 F and 104 F (22 C and 40 C) is shown in the graph (see Fig. 1 which is reproduced from Uhlig). Specimens of mild steel were exposed to water containing an average of 5 ml of O_2 per liter; *NaOH* and *HCl* were added in proper amounts to produce the respective alkaline and acid ranges shown. At values of pH greater than needed for hydrogen evolution (pH = 4.0) the results of the graph can be explained by formation of a protective layer of hydrous ferrous oxide on the iron surface, from the initial corrosion reaction. Regardless of the actual pH of the water between pH 4.0 and 9.5, the surface of the iron is always in contact with an alkaline saturated solution of hydrous ferrous oxide (pH approximating 9.5). Corrosion continues under these conditions only as rapidly as O_2 can diffuse through the protective layer. The layer, moreover, is continually being renewed by the corrosion process. Since this corrosion product film next to the iron is essentially unchanged by external conditions

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†See also ACI JOURNAL, Sept. 1950, *Proc.* V. 47, p. 74 and Jan. 1951, *Proc.* V. 47, p. 397.

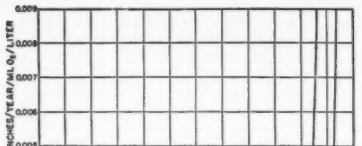

Fig. 1—Effect of pH on corrosion of mild steel

(Reproduced by permission from *Corrosion Handbook* by H. H. Uhlig, published by John Wiley & Sons, Inc., 1948)

within the above range of pH, the corrosion rate is not altered except by a change occurring in the amount of the dissolved oxygen. At pH 9.5 (22 C), however, a further increase in the alkalinity will extend its effect to the iron surface and there either decrease the surface reaction rate (cause the iron to become passive), or decrease the permeability of the reaction product layer to oxygen, *i.e.*, precipitate the protective hydrous *FeO* in a dense film very close to the steel surface. In either instance the corrosion rate will be decreased, as has been observed . . . "

Evans[2] says: "The behavior of steel, embedded in concrete, towards stray currents differs from that of bare steel in two respects. The alkaline character of most concretes will reduce the danger of attack setting in, but if it does set in, the damage will be serious, since the voluminous rust produced will crack the concrete.

"Calcium hydroxide is liberated during the setting process of portland cement and the alkaline reaction usually keeps steel buried in such cement passive. However; the presence of too much free calcium hydroxide in portland cement (or in concrete) may be detrimental to the permanence of the material, since the hydroxide may be leached out by water, or altered to other substances which occupy different volumes; both of these changes favor disintegration. It is better, therefore, not to aim at a mass which will be exceptionally alkaline after setting, but rather to aim at a mass which will be as resistant and watertight as possible. . . . The destruction of concrete blocks by the expansive forces of corroding iron has been studied in the laboratory, where it was found to produce stresses as high as one ton per sq in."

It may be noted from Uhlig's graph that the specific penetration of oxygen causing a corrosive attack upon the steel at 22 C drops about 37 percent between pH 7.0 and pH 11.0, which is about the pH of a slurry of water and cement that has stood for some time with occasional stirring to bring about equilibrium between the phases.

Bogue[3] gives typical compositions for various types of portland cements on the market. He also reports the amount of lime released in from six months to two years from tricalcium silicate and dicalcium silicate; compounds in the cement before hydration.

There is no lime released from the other compounds in the cement. Analyzing the compositions given by Bogue on the basis of ASTM C 150 we have compounds formed (Table 1) which, according to Bogue, result in the indicated percentages of the weight of the two silicate compounds, and in addition, based upon the total weight of cement being freed as *CaO*.

If, then, we have a 1:6 cement-aggregate concrete weighing 150 lb per cu ft, the cement will approximate 11 percent by weight of the concrete. The extractable *CaO* freed from the various types of cement will then be in percent by weight of concrete:

Type I —1.08
Type II —1.08
Type III—1.25

If now we assume that 1.31 g of *CaO* can be dissolved as $Ca(OH)_2$ by each liter of water percolating through the concrete, it will be found that it will require in the neighborhood of 20 cu ft of percolating water to extract the lime freed from the two cement compounds in each cubic foot of concrete. The amount which can actually be extracted by a given quantity of percolating water will, of course, depend upon the percentages of C_3S and C_2S present in the cement, and also upon their degree of hydration.

TABLE 1—FORMATION OF COMPOUNDS IN CONCRETE

	Type I	Type II	Type III
C_3S, percent of the cement	46	45	56
C_2S, percent of the cement	26	30	16
CaO freed from:			
C_3S, percent of C_3S	19.3	19.3	19.3
C_2S, percent of C_2S	3.7	3.7	3.7
CaO freed from:			
C_3S, percent of cement	8.88	8.69	10.81
C_2S, percent of cement	0.96	1.11	0.59
Total CaO freed, percent of cement	9.84	9.80	11.40

However, some of the CaO released from the hydrating mineral may be converted to $CaCO_3$ by CO_2 dissolved in the air or water coming in contact with the concrete; a saturated solution of $CaCO_3$ in water denuded of its CO_2 (this being the condition of water in contact with the usual concrete because of the reaction $Ca(OH)_2 + 2H_2CO_3 \rightleftarrows Ca(HCO_3)_2 + 2H_2O$) may be said to have a pH between 8.0 and 9.0. These considerations lead to the conclusion that as long as the concrete contains free CaO or $Ca(OH)_2$ steel imbedded in the concrete will corrode at a reduced rate. If, however, the CaO be removed either by leaching or by conversion to the carbonate, corrosion will go on about as it would in an atmosphere exposure, except as the envelope about the concrete may reduce the rate at which the water and oxygen can penetrate to the steel.

Reduced corrosion rates at pH 10.0-11.0 may be ascribed to the lowered solubility of the ferrous ions in an alkaline solution causing their precipitation as they leave the steel to form a close clinging tight film of hydrated ferrous oxide upon the surface; the precipitated oxides then act as a more or less impermeable film to restriet the passage of water and oxygen to the steel, or partially polarize the anodic and cathodic areas. Concrete then will protect steel or reduce its rate of corrisoon in two ways: (a) by excluding oxygen and moisture from the steel, *i.e.*, by lowering the permeability to passage of these corrosive agents; and (b) because the CaO is more slowly leached out by the water or converted to $CaCO_3$. The longer the alkaline environment can be made to continue, the longer will the protection be effective.

Steel too near the concrete surface when exposed to air and water will rust at a greater rate than will that which is more deeply imbedded because of the greater penetration of water and oxygen, and this greater rate of rusting will be evidenced by spalling of the concrete adjacent to the steel because of expansion of the iron hydroxides.

The tests of McClenahan, Beck and Palmieri[4] and some tests made over a period of years in the physical laboratory of the Sanitary District indicate that the addition of fly ash makes the concrete more impermeable to water, air and carbon dioxide and their aqueous solutions. It should therefore reduce the corrosion rate of steel imbedded therein below that of an ordinary atmospheric exposure and below exposures where the steel is imbedded in more porous concrete.

Davis[5] found that one of the outstanding characteristics of concretes containing fly ash was the decrease in permeability over that of concretes made without fly ash. That this decrease in permeability is due to any marked increase in the density is to be doubted, although the concrete often is more dense. We are inclined to think that the impermeability comes about more particularly from the nature of the gel which is formed when fly ash is present.

In a low carbon fly ash sulfur may be presumed to be present mostly in an oxidized condition with possibly minor amounts combined with iron as an iron sulfide. Davis[6] reports the sulfur content of 20 samples of fly ash from different parts of the United States calculated as SO_3. The average content shown is 1.20 percent, the maximum is 3.26 percent, and the minimum 0.01 percent.

The average SO_3 in 29 samples of Type II portland cement used in the Sanitary District of Chicago's construction was 1.77 percent. It would seem, therefore, that the fly ash would cause little, if any, more corrosion than would an equal volume of portland cement and would be no great factor in influencing the corrosion rate so long as the pH stayed above 6.0 or 7.0. Furthermore, in the Evans[7] experiments on corrosion rates where solutions of calcium and magnesium sulfates were used, the corrosion rates were less than they were in the presence of distilled water. Also it should be noted that the sulfate ion is less active generally as a promoter of corrosion than is the chloride

ion. The iron sulfide compounds, moreover, should not be expected to free any H_2S to attack the steel so long as the pH remains above 6.0. Users of fly ash, such as the U. S. Bureau of Reclamation, specify that the maximum SO_3 shall not exceed 3.0 percent which is well above that found in most well burned fly ash, particularly in the midwest.

Carbon in fly ash may be much more of a problem as regards corrosion than is the sulfur. Moreover, the carbon is usually responsible for other undesirable properties which are often entailed or predicated by its presence as found by Davis.[8]

The Sanitary District, in some recent contracts where fly ash has been specified, has limited the carbon to a maximum content of 3 percent. Since the fly ash itself does not usually exceed 25 percent by volume of the cementitious material (21 percent by weight —the Sanitary District uses 16⅔ percent by volume), which amounts to 2.3 percent by weight of the concrete, it would appear that the 3 percent carbon, based upon the weight of the fly ash, would be such a small amount in the concrete and be so well dispersed through the mass that the conductivity of the concrete would be hardly affected at all, which would be necessary if the corrosion of the steel is to be increased.

Summary

1. The alkaline condition which prevails within concrete so long as the lime is not leached out tends to maintain a protective film of ferrous hydroxide upon the steel surface. This film prevents the easy penetration of water and oxygen to the steel to further corrode the surface. Fly ash does not materially change this alkalinity.

2. Fly ash, by its pozzolanic reaction, seems to increase impermeability of concrete, thereby decreasing penetration of oxygen and water to the steel. The pozzolanic gel also seems to decrease the amount of the lime which can be leached out of the concrete. This may be due entirely to a physical resistance to the passage of the water due to its peculiar structure, or to a chemical fixing of the lime as some investigators believe.

3. Sulfur compounds in fly ash are usually so limited by specifications that they are not materially different in the concrete, whether fly ash is used or not. Moreover, the alkaline condition in the concrete is unfavorable to a sulfate attack on steel.

4. Carbon in fly ash would appear by theoretical considerations to be much more significant in concrete than is sulfur. The actual effect should be investigated. However, if it is kept under 3 percent in the fly ash its percentage in the concrete becomes so small that if it is well dispersed its effect on the electrical conductivity of the concrete, and therefore upon the corrosion of the steel, should be quite minor. If, however, the carbon should become segregated or be in lumps it might become a serious source of trouble. When not over 3 percent is permitted in the fly ash, however, it usually is so finely divided and dispersed that few lumps or segregations will occur.

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Skepticism Regarding the Plastic Theory of Design (LR 47-66)*

Homer M. Hadley's entertaining letter[1] expressing his doubts as to the existence of short-time plastic flow crystallizes a confusion in thinking which still exists among some engineers concerning the basis and reasoning underlying the proposed plastic theories. For this reason a serious reply is deserved.

It is surprising to note his complete lack of recognition of the past and present research which substantiates these theories. It is impossible to reconcile his statements that the plastic theories are purely speculative with the abundant data on concrete behavior. Reports describing the effect of short-time flow have been available for many years. The writers are familiar with a partial bibliography on plastic flow which lists over 80 individual papers published since 1900.

A good plastic theory was proposed in the latter part of the 19th century, and almost all textbooks since that time include explanations of plastic behavior at high stresses and state that a law of stress variation which more nearly agrees with the facts (than the straight-line theory) is necessary to obtain correct results in flexural computations.[2] Prominent research organizations and agencies† manned by highly competent personnel with professional recognition have prepared careful and informative reports of the flow characteristics of concrete, and, as far as is known by the writers, all these groups are in substantial agreement on the specific characteristics influencing long and short time flow as well as the reality of its existence.

It is generally accepted that creep continues for years though the rate of creep decreases with time and hydration of the cement. The rate of creep also is roughly proportional to the stress except at the critical loads. Tests indicate that there is a flow in tension even greater than that in compression. Hadley's concept of plastic flow as a function of time only disagrees with test data which indicate that the rapidity and magnitude of flow is a function of the applied stress, other factors being equal. Tests[3] show that the rate of plastic flow increases sharply as the load is increased above about 75 percent of the standard ultimate strength although loads from 85 to 90 percent of ultimate strength can be maintained permanently despite extremely large strains.

While some minor disagreement may exist among research groups as to the formulas best expressing the effect of this behavior in the practical design of reinforced concrete members, these variations produce essentially the same results. Recognition of the plastic behavior of concrete has already culminated in the acceptance of plastic theory design methods in codes in force in South America, Russia and some European countries.

Besides the work already published, the results of many additional research projects now underway or which have already been completed offer much more complete information on certain phases of plastic behavior.

It must be remembered that the choice of either the straight-line or the plastic theory depends on practical considerations and that the primary purpose of any theory is to provide a tool by which the useful strength of individual concrete members may be computed and the most desirable proportions of assembled members can be established.

The conventional straight-line method, unfortunately, is lacking in this respect. In flexure, the near-failure load gives values of compressive strength more than 1.50 times that indicated by the cylinder strength. An empirical increase in the cylinder strength, as suggested by Hadley, is not a suitable solution, for while this might result in more economical use of the concrete it would give incorrect values for the lever arm and would in many cases provide reduced safety against overload. The use of varying coefficients for different strength concretes and possibly additional coefficients to account for the effect of beam dimensions[4] and different percentages of steel reinforcement as indicated by test would make the straight-line method even more impractical than at present. The additional confusion which could result may be inferred from the practice in certain European countries where the specifications allow three different values of compression stress as a function of beam dimen-

*See also ACI JOURNAL, Nov. 1950, *Proc.* V. 47, p. 257 and Jan. 1951, *Proc.* V. 47, p. 398.

†U. S. Bureau of Reclamation, Dept. of Scientific and Industrial Research, England, University of Illinois, Ohio State University, Massachusetts Institute of Technology, Queens University and many others.

sions for each value of cube strength, allow increased stresses in the webs of T-beams, require a constant design value of $n = 15$ for all classes of concrete and a smaller value for computing deflections.

That the straight line theory has been abandoned for axially loaded members and is used in a modified version for flexural members with compression steel speaks for itself. Moreover, the writers do not believe that the long-time plastic flow in columns determined the choice of plastic design methods for these members. Many short time tests, in addition to those mentioned by Hadley, show the typical plastic behavior under short-time loads. Practically all codes specify the use of a plastic theory in column design despite the fact that the short-time live load is an important part of the total design load and probably the entirety of overloads.

Because the test of the practicality and realism of any procedure is its utility, generality and simplicity, the relative merits of plastic theory and straight-line theory should be examined in light of these requirements.

As admitted by Hadley, the plastic theory does make excellent predictions of the ultimate structural capacity of reinforced concrete members. The plastic theory is also general in use, applying equally well to members subject to flexure, to combined direct stress and flexure, and to axial loads without flexure.

Simplicity is perhaps the greatest requisite. The plastic theory fulfills this requirement, being both simpler and more rapid in application than the straight-line method.

While much remains to be learned about the behavior of concrete, observation of large scale action of various members under test conditions and microscopic examination of the material itself has led to a better understanding of its action under stress. Concrete may be described as a very nearly visco-elastic material consisting of a skeleton of solid, practically elastic aggregates united by a viscous cement paste.[5] The viscosity of the cement paste allows non-destructive flow in the concrete. After a time, partial crystallization of the viscous paste reduces and finally halts the flow. This phenomenon has been verified by numerous observations which note a decreasing rate of creep with time. If the creep becomes sufficiently large the particles of the skeleton come into contact, resisting load by direct bearing of the skeleton particles as well as by the resistance of the viscous cement filler. Further motion causes local rupture and failure and a relaxation of the load to other strong points. It is apparent that sufficiently large movements inevitably will lead to deterioration and failure. The damaging effect of the creep at any stage, however, can be evaluated only if the inelastic strains due to rupture of the framework can be separated from the harmless viscous deformations.[5]

As might have been expected, Hadley's test cylinders[6] were found to be shattered at the extreme end of the descending branch of the stress-strain curve. Since the strains at that time were between 0.004 and 0.006 in. per in., the writers fail to understand why he was apparently surprised to find that the concrete was then "devoid of structural significance." He might have arrived at different answers had he unloaded, examined and reloaded the cylinders at any time before signs of failure appeared.

In describing the test of a high strength cylinder Hadley reports the complete absence of any measurable plastic range. There is, however, an inexplicable discrepancy between two of the curves with which he has attempted to analyze his data. This the writers believe to be the result of confusion between time and space. One curve indicates an abrupt drop in load accompanied by an equally swift increase in deformation as a function of time. The corresponding stress-strain curve shows no increase in strain during the drop in load. Since failure of concrete can be related primarily to excessive strains, the writers feel that this curve too should have had a descending branch with a large ultimate strain at total failure. The inability to observe this large plastic strain is probably the result of elastic deformations of the testing machine and the sudden release of energy stored in that deformation.[7]

Though the point at which permanent damage occurs is difficult to forecast, certain tests indicate limits of strain which do not appear to affect either the strength or the integrity of the concrete. Of these, the tests of continuous beams and frames by Glanville and Thomas[8] are of great interest.

These members were loaded until capacity moments were developed at both the center points and the points of continuity at strains which were in excess of those contemplated in the derivations of the plastic theories.

Observations of those parts of the test members strained far beyond their elastic capacity led the authors to the following conclusions: (1) The concrete continued to carry the load with little increase in moment capacity long after the strains had passed the peak elastic value. (2) No distress was apparent when the concrete yielded. (3) The concrete was highly ductile in behavior. (4) The capacity of concrete to yield enabled the beam to take advantage of the increased load-carrying capacity incident to the distribution of excess moment to other parts of the redundant members.

These conclusions indicate the existence of a descending branch of the stress-strain curve.

Glanville and Thomas came to the conclusion that failure was a condition of strain and that this strain would be approximately the same for high strength as for low strength concretes. From their observations they believed that distress would occur at strains of about 0.003.

Tests by R. H. Evans[9] showed a similar effect. In his tests the position of the neutral axis at low loads agreed with the standard straight-line theory, including tension. With increased load the neutral axis rose correspondingly to the point predicted by the standard theory without tension in the concrete until the strain approached 0.001. For larger strains, and especially when the strain exceeded 0.002, the neutral axis began to fall. The fall in the neutral axis for members failing in compression was considerable. During this time, measurements showed that the strains in the compression side of the beam increased almost in direct proportion to the distance from the neutral axis.

The straight-line theory can not account for this behavior or similar action observed in other tests. In contrast, this phenomenon does agree with the predictions of plastic theory, thus providing a means for correlating various empirical solutions and for preparing a generally acceptable description of plastic action. However, the necessity for a description is academic; the validity of a method depending primarily on the accuracy of formulas in predicting the results obtained from the tests. For example, if the descending branch of the stress-strain curve is neglected, the compression area and the lever arm would be little changed. For any description of stress distribution an equivalent stress rectangle having some fixed ratio to cylinder strength could be substituted to give the same total moment as the exact curve. If this rectangle satisfies the requirements of design for all types of members it would become an acceptable design tool.

Whitney, in the articles describing his proposed plastic theory, states that he chose 85 percent of the standard cylinder stress as the abscissa of the equivalent rectangle because this value agreed with accepted test values for concrete columns and also gave results which agreed consistently with tests for members subject to flexure or combined flexure and axial load. The coefficient of $0.85f'_c$ accounts for the distribution of the compressive stress and the ratio of cylinder to beam strength at conventional rates of test loading. Hadley is mistaken when he states that the rectangular stress block proposed by Whitney represents an assumed stress distribution, or that the bottom of the rectangle is the neutral axis.

The writers do not agree that the behavior of concrete in cylinders or cubes is essentially different than its behavior in beams or columns—other than minor differences caused by somewhat different boundary conditions or by behavior described by plastic theory. The cylinder strength may be closer to the beam strength than is usually believed because at the time of the maximum moment the stress may be relatively uniform over most of the compressive area rather than varying linearly as is assumed by the straight-line theory. The ultimate stress in the cylinder and the ultimate stress in the beam may vary considerably if the rate and duration of the loading are changed. The same applies to a change in the other factors influencing plastic flow. However, the rate of change of flow under the higher unit stress balances the effect of the other variations and as a result cubes and cylinders tested at a specified age and a specified rate of loading provide a simple and economical comparison for evaluating concretes of various mixes and consistencies. Because of the variable relationship

between cube and cylinder strengths and the better correlation of cylinder strength to beam tests, cylinder tests provide a somewhat better control basis for the plastic theories.

The writers also object to Hadley's statement that "of course, in full size structures, failure-producing loads follow through and apply undiminished in exactly this same way (as in a testing machine)." It appears to the writers that such is only occasionally the case and then only for the columns of a simple structure. For example, in continuous structures of reinforced concrete it has been adequately demonstrated by Glanville and Thomas[8] that extremely large deformations must take place before failure occurs. During this deformation the load is *transferred* from members whose strength has been exceeded to those having reserve capacity.

This is actually the second step in load redistribution, for in the flexural members themselves the concrete in the tensile zone has already transferred its load to the reinforcing steel, and the extreme fibers in the compressive zone have sloughed off the stresses prescribed by the straight-line theory. The latter must be admitted even by Hadley since the cylinders and curves which he has presented do not show failure until wide departures from elastic action have taken place. If cylinders were tested under similar conditions of relief from overload the true descending branch of the stress-strain curve could be determined.

Continued research may or may not make any appreciable changes in the coefficients advanced for the practical application of plastic theory. It undoubtedly will eliminate many minor variations which now exist and indicate the form most useful to practicing engineers. Although the straight-line theory has produced many sturdy structures, it cannot be said that these structures are economical of material or of consistent strength. Their proportions often lack grace because of artificial restraints placed on designers by the false assumptions of conventional straight-line design procedures.

Because many of the defects of current design practice could be eliminated by plastic theory, even without further refinement, and because it has already received wide acceptance and verification, the writers believe that American engineers should give serious consideration to incorporating a plastic design method into the building codes of the country at this time when the most efficient utilization of our resources is necessary.

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BOYD G. ANDERSON and EDWARD COHEN, Ammann and Whitney, Consulting Engineers, New York, N. Y.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Granolithic concrete floors

Digest No. 24, Nov. 1950, Building Research Station, Garston, Watford, Herts, England, 4 pp.

Mix proportions and techniques for laying subfloors are given. For the topping a 1:1:2 mix is recommended. Precautions in using admixtures and pigments are mentioned. Procedures for laying and finishing the wearing course are described.

Design of rectangular and triangular reinforced concrete ribbed plates (Contributo al calcolo delle piastre di cemento armato rettangolari e triangolari, vincolate sul contorno)

A. Danusso, *Il Cemento* (Milan), V. 47, No. 6, June 1950, pp. 96-100

Reviewed by Gennaro Mianulli

This is the tenth in a series of articles on the analysis of ribbed plates. The author's contribution is based on the theory and tests developed by Bach.

Structural theory and design, V. 1

J. McHardy Young, Crosby Lockwood & Son, Ltd., London, 1950, 286 pp. 25 shillings

Intended for the undergraduate student or the young engineer preparing for professional examination many of the numerical examples are based on questions from British examinations. While the book deals with the methods of rigorous analysis the examples indicate the application of first principles to practice. The worked examples stress that the designer must acquire the necessary skill to modify theoretical requirements to suit practical design problems.

Torsion in reinforced concrete structures (Sulle sollecitazioni di torsione nelle costuzioni in cemento armato)

E. Castiglia, *Il Cemento* (Milan), V. 47, No. 6, June 1950, pp. 104-105

Reviewed by Gennaro Mianulli

Castiglia continues his theoretical treatment of the stresses developed by torsion on marginal beams and illustrates the application of his findings with a few practical examples.

Rapid analysis of arch bridges (Calcolo rapido dei ponti ad arco) Part 2—Fixed Arch–General Case

Domenico Priolo, *Giornale Del Genio Civile* (Rome), V. 88, No. 2, Feb. 1950, pp. 103-113

Reviewed by Gennaro Mianulli

In this treatment, a hingeless arch is proportioned and its stresses determined by adopting its shape and dimensions according to the law of variation of the moment of inertia from crown to skewback.

Foundations for structures storing high temperature fuels (Fondazioni di apparecchi ad alta temperatura)

C. Lamparelli, *Il Cemento* (Milan), V. 47, No. 5, May 1950, pp. 89-91

Reviewed by Gennaro Mianulli

A structure storing high temperature fuels induces by expansion secondary stresses in its reinforced concrete foundation. These stresses are serious for temperatures above 100 C (212 F), and are determined on the basis of few assumptions which simplifies the design.

*A part of copyrighted Journal of the American Concrete Institute, V. 22, No. 6, Feb. 1951, *Proceedings* V. 47. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

Rigid frame structures computed without solving linear equations (I telai piani semplicemente connessi cacolati senza equotioni)—Part 1

BRUNO DALL'AGLIO, *Giornale Del Genio Civile* (Rome), V. 88, No. 2, Feb. 1950, pp. 93-102
Reviewed by GENNARO MIANULLI

The author attempts to simplify the analysis of multi-span rigid frames by using the theory of the ellipse of elasticity and by determining the ideal geometric characteristics of two legs of each bent.

Simplified formulas for strength computation of rectangular reinforced concrete beams without the use of tables (Formule simplificate per il calcolo delle sezioni rettangolari inflesse di cemento armato)

VITO CAMIZ, *Giornale Del Genio Civile* (Rome), V. 88, No. 3, Mar. 1950, pp. 141-144
Reviewed by GENNARO MIANULLI

The author bases all his computations on three simple formulas, two of which are approximate. The error is negligible, but the use is limited to rectangular beams with tension reinforcement only.

Retarding agents produce unstable equilibrium (L'instalilita' dell' equilibrio in regime elastico-viscoso)

FRANCESCO LEVI, *Giornale Del Genio Civile* (Rome), V. 88, No. 1, Feb. 1950, pp. 84-92
Reviewed by GENNARO MIANULLI

This is a theoretical analysis treating four cases of equilibrium instability produced by retarding agents: an un-perfect straight bar, loaded axially; a bar simultaneously subjected to axial and lateral loads; bar fixed at ends (redundant), loaded axially, and a nonperfected circular ring, subjected to uniform pressure.

Mechanically determining the time of set of portland cement by means of the spissograph

O. J. GLANTZ and L. E. HALSTED, *ASTM Bulletin*, No. 171, Dec. 1950, pp. 79-81

In an effort to relieve an operator of the responsibility of intermittently checking time of set on cement samples, an automatic machine has been devised. The test results include the conventional hand method and the mechanical method, both on neat cement and mortar. The conclusion is that it is possible to do a satisfactory job of mechanically determining time of set, particularly on mortar samples.

Precise measurements of deformation in compression tests (La mesure precise des deformations dans les essais a la compression)

R. M. BERTHIER, *Revue des Materiaux* (Paris), No. 421, Oct. 1950, pp. 301-305
Reviewed by PHILLIP L. MELVILLE

As in the United States in recent months the question of plasticity *vs* elasticity of hardened concrete again is being raised elsewhere. A special device was used to test concrete cubes in compression. Precise and complete stress-strain curves were obtained. Interesting data were obtained on Poisson ratio, Young's modulus, breaking loads and limits of flow.

The principles of making high-strength concrete

A. R. COLLINS, *Constructional Review* (Sydney), V. 23, No. 5, Sept. 1950, pp. 22-30

The general factors influencing concrete strength are outlined and their variation in the high ranges are discussed. Mix proportions and methods of compaction and curing are suggested for 4000 to 11,000 psi concretes. Good quality materials, carefully controlled and batched with the mix proportions chosen to give an average strength sufficient to allow for expected variations, suitable curing condition and careful supervision of the work are given as essentials to the production of high-strength concrete.

Study of clinker bricks in rotary cement kilns (Etude des briques de clinker dans les fours rotatifs a ciment)

L. HALM and P. LAPOUJADE, *Revue des Materiaux* (Paris), No. 420, Sept. 1950, pp. 273-81; No. 421, Oct. 1950, pp. 305-12
Reviewed by PHILLIP L. MELVILLE

Refractory bricks are made of bauxite in Europe or bauxite and aluminum monohydrate in the United States. Clinker protects the bricks by an insulating layer. Questionnaires were sent to cement manufacturers. Used bricks were carefully inspected and submitted to X-ray, chemical and microscopic studies. It appears that the formation of free lime in the center causes failure through hydration when the kiln is allowed to cool. Checks on brick temperatures around 1000 C were made with thermocouples. Laboratory tests were performed as a basis for recommended changes in grain size, percentage of cement and method of manufacture. It is suggested that active silica be added. Eleven references are included.

Plant engineering handbook

WILLIAM STANIAR, Editor-in-chief, McGraw-Hill Book Co., Inc., New York, 1950 pp. $15.00

Handbook inadequately indicates the mass of information compiled by the editor and 86 contributing authors over a period of five years. It is designed to meet the needs of plant managers, superintendents and maintenance engineers. The book deals with industry's consumption and conservation of basic resources, management engineering, industrial construction, fire protection, maintenance, industrial power, air and materials handling. Specialized subjects of instrumentation, bearings and lubrication, engineering materials and mechanical power also are presented.

Settlement correction at La Guardia field

JOHN M. KYLE, *Proceedings*, ASCE, V. 76, Separate No. 43, Nov. 1950, 6 pp. $0.50

Portions of La Guardia Field at New York City composed of fill over soft silty clay settled as much as 5 ft resulting in inundation of the runways and interruption of operations by high tides under the influence of northeast winds. Sand drains were tested but abandoned in favor of an earth dike around the low areas. Ditches and sump pumps provided field drainage.

At the time this report was written the dike had permitted uninterrupted operation of the field. However, the extreme storm conditions of November 25, 1950, halted operations when the field was again flooded.

Re-erection of the Ruther Leine bridge damaged by flood in 1946 (Wiederherstellung der durch Hochwasser 1946 zerstoerten Ruther Leinebruecke)

W. MEINERS, *Beton-u. Stahlbetonbau* (Berlin), Apr. 1950, V. 45, No. 4, pp. 73-77

Reviewed by RUDOLPH FISCHL

This reinforced concrete girder bridge was built in 1928-29 with three equal spans, each 85 ft long and 28 ft wide. During a long-lasting heavy flood in the spring of 1946 the pier in the high-water channel was undermined, broke and sunk about 8 ft. The three continuous main girders broke in the exterior spans near the piers.

The sunken bridge was lifted at three points and a new pier built to replace the broken one. Then all the damaged parts of the concrete in the girders and deck were carefully removed and the reinforcement exposed. The bent-up bars were straightened and the broken bars are welded. New concrete was placed in these repaired parts, thus re-establishing the continuity of the three spans.

The total cost for this reconstruction job was far below that estimated for building a new deck and clearing away the debris.

A study of the determination of the heat of hydration of portland cement

EDWIN S. NEWMAN, *Journal of Research*, National Bureau of Standards, V. 45, No. 5, Nov. 1950, pp. 411-417

AUTHOR'S SUMMARY

Three relatively inexperienced operators made six determinations each of the heat of hydration of a sample of portland cement. The results of their measurements were calculated by the method described in Federal Specification SS-C-158b and by two simpler reduced-observation methods. By statistical analysis, no significant differences were found among either operators or methods. The precision (standard deviation of a single determination) of the heat-of-hydration measurements was about 3 calories per gram at 7 days and 2 calories per gram at 28 days. Somewhat better precision can be expected from experienced operators.

Symposium on monolithically-cast concrete buildings

Cement och Betong, (Malmoe, Sweden), V. 25, No. 3, pp. 135-228

Reviewed by EIVIND HOGNESTAD

Monolithically-cast buildings are constructed with concrete slab floors and load-carrying walls and partitions. This type of construction was used in Sweden just before World War II. In 1939, 60 large concrete buildings were planned in Gothenburg of which 58 were designed as monolithically-cast structures.

Some advantages of such structures are: excellent sound insulation, good thermal insulation, good fire resistance, effective protection under air bombardment, considerable gained space, short construction time, and more freedom in architectural design.

Due to the lack of materials during the war, the development of this type of construction was retarded. Through a recent book "Monolithically-Cast Concrete Build-

ings," the Swedish PCA encourages a return to this promising prewar structure.

The present symposium discusses various phases of such structures through eight articles, most interesting of which may be a description of slip-forms used in the construction of apartment buildings.

Specifications for the design of prestressed reinforced concrete members (Vorgespannte Stahlbetonbauteile, Richtlinien fuer die Bemessung)

Beton-u. Stahlbetonbau (Berlin), V. 45, No. 4, Apr. 1950, pp. 80-90

Reviewed by RUDOLPH FISCHL

The German committee for reinforced concrete presents the seventh outline of DIN 4227, on which work started in 1943. Members of the committee are well-known experts, including Bornemann, Dischinger, Hummel, Nakonz, Wedler, Rüsch and others.

The specifications are presented in 16 paragraphs treating quality of materials, methods of prestressing, fundamentals of design, creep and shrinkage, working loads, safety against cracks and failure, shear and bond, buckling, anchorage of the prestressed members, allowable stresses, and many others.

The reinforcement is classified in four groups: plain or deformed bars, cold drawn wires and cables, spiral-wound wire ropes with one strand, and spiral-wound wire ropes with several strands. Minimum reinforcement is limited to 0.2 percent.

This reinforcement has to be used with high quality concrete only. The modulus of elasticity, and the *n*-ratio for the different qualities of concrete and steel, and coefficients for creep and shrinkage are given in tables. The computation for safety against failure is especially emphasized. A table with the allowable unit stresses concludes the specifications.

Interpretation of the specifications for the design of prestressed reinforced concrete members (Erlauterungen zu den Richtlinien fuer die Bemessung vorgesponnter Stahlbetonbautiele)

H. RUSCH, *Beton-u. Stahlbetonbau* (Berlin), V. 45, No. 5, May 1950, pp. 108-116

Reviewed by RUDOLPH FISCHL

Professor Rüsch of the Technical University in Munich, who deserves special mention for his collaboration with the committee responsible for these specifications, wrote this interpretation to facilitate the understanding of the proposed code.

Every paragraph is explained carefully. The essentials are pointed out and the reasons for certain formulations are given.

This paper is an excellent addition to the specifications and should not be missed by anybody studying the proposed code.

Lectures on reinforced concrete (Cours de beton arme)

ADRIEN PARIS, Dunod, Paris, 499 pp. 4350 Fr. (about $12.50)

Reviewed by PAUL ROGERS

This is the second volume of lectures on reinforced concrete by one of the foremost French authorities on the subject. The author is professor emeritus of the Technical University of Lausanne, Switzerland, and his present volume comprises his lectures in the graduate school.

The first chapter is devoted to the fundamentals of advanced structural theory. Elasticity, deformations, reciprocity of elastic deformations, least work, Castigliano's theorems, classical analysis of indeterminate structures, flexural and shear deflections, curved members, and similar fundamentals are analyzed. Chapter 2 introduces the concept of the ellipse of elasticity (or of stress), while chapter 3 deals with dynamic problems and the theory of vibrations.

In later chapters earthquake shocks and their effect on structures are analyzed and dynamic and vibration problems for bridges are presented. Two chapters deal exclusively with airport runways and taxiways. Both uniform and nonuniform temperature changes in the runway-slabs are discussed, and the slabs as beams on elastic foundations are treated. The instantaneous impact effect of a landing plane is analyzed.

Dynamic forces acting upon machinery foundations are considered. Vibrations, resonance, oscillations of foundation supporting piles are investigated together with suggestions as to the design of foundations.

One chapter takes up the effect of bombardment on structures; the penetration of bombs and their explosive energies are calculated, and design analyses for bomb-blast resistant structures are presented. The last chapter is devoted to the problem of monolithic concrete bridge floors.

Civil engineering handbook

LEONARD C. URQUHART, Third Edition, McGraw-Hill Book Co., Inc., New York, 995 pp. $8.50

This enlarged edition contains ten major sections giving theory, practice and fundamentals of civil engineering. Rules, principles, pointers and working instructions are given for surveying, railway, highway and airport engineering, mechanics of materials, hydraulics, stresses in framed structures, cement and concrete, foundations and other fields.

The section on concrete has been almost completely rewritten by Herbert J. Gilkey, ACI past president. A brief discussion of nonhydraulic cementing materials and hydraulic cements other than portland gives data on pozzolanic and natural cements. Portland cement is more extensively treated. A short historical discussion is followed by description of the various types currently available. ASTM acceptance tests are summarized.

The characteristics of concrete as an engineering material are considered and the effect of various properties and conditions of manufacture are assessed. Mix design and selection and grading of aggregates are more extensively covered. A section on practical aspects contains valuable hints for the field man.

Under reinforced concrete, design for flexural loading, columns, walls, footings and foundations, retaining walls and abutments, arches, box culverts and rectangular frames are included. Excerpts from the ACI Building Code are included to show their application.

Eighty-four specific references are given as well as a list of organizations doing important work in the field.

Excellent sections on surveying by Harmer E. Davis and on soil mechanics and foundations by A. E. Cummings are valuable additions to the handbook.

Cooling aggregates and cement for Detroit Dam

WALTER B. LENHART, *Rock Products*, V. 53, No. 10, Oct. 1950, pp. 90-97

Reviewed by DONALD M. AGRIMSON

Methods and equipment used at Detroit Dam to satisfy the rigid specifications that call for concrete to be placed at 40 F minimum and 50 F maximum.

Detroit dam is 50 miles east of Salem, Ore., and is being constructed across the North Santiam River, a tributary of the Willamette River. The structure will contain 1,500,000 cu yd of concrete.

This is the first major project where all aggregates as well as the portland cement are cooled. The intent is to have the coarse aggregates arrive at the mixers at 38 F, the sand 50 F, and the portland cement at 60 F. It is expected that the cooling system will be able to deliver materials at the above temperatures even though portland cement arrives at 150 F, sand at 70 F, and coarse aggregate at 65 F.

Briefly, cooling is accomplished as follows:
1. Sand and portland cement will be cooled by specially designed screw-type conveyors with cold water flowing countercurrent through hollow steel flights. Sand will be conveyed through four flights and portland cement through three flights. The author points out that some mechanical features of the above system are new and untried and that many "bugs" may have to be worked out.
2. Coarse aggregates are cooled by the immersion method. There is an immersion tank for each size aggregate, plus one spare. Water can be circulated through the aggregate for any length of time desired. The time interval of immersion can be preset so that the tanks will drain automatically after the required period of cooling.

Heat insulation of flat concrete roofs (Varmeisolering af flade betontage)

B. J. RAMBOELL, *Beton og Jernbeton* (Copenhagen), V. 1, No. 3, Dec. 1949, pp. 93-102

AUTHOR'S SUMMARY

Concrete roofs covered with a layer of waterproof material such as asphalt or tar paper are often found to contain considerable quantities of moisture. Often the water drips down into the rooms below. Even under undamaged waterproof roofing the concrete is often found to contain much more moisture than immediately after finishing.

The moisture comes from inside the building. During winter the warm indoor air will be able to absorb more moisture than the cold air outside, and the vapor pressure is therefore highest indoors, where the humidity is high on account of respiration, cooking, etc. In factories with exceptionally

high humidity the vapor pressure is so high that the roof becomes saturated with moisture, which may drip down into the rooms below, and in any case will reduce the roof's insulation considerably.

A roof can be kept dry by ventilating it with a stream of air. The air passes through the construction parallel with the roof's surface and carries away the moisture which has passed into the roof.

A nonventilated roof over a relatively dry building may stay dry. Presumably partly because during the summer when the vapor pressure often is highest in the external part of a roof exposed to the sun, moisture moves from the roof towards the rooms underneath, and partly on account of capillary forces which counterbalance the movement of moisture in the opposite direction.

Several examples of ventilated and nonventilated roofs are shown. Few data from field and laboratory experiments are available, but it seems probable that sooner or later the conclusion will be drawn that a certain amount of ventilation—more or less according to the humidity inside the building—should always be provided.

Proceedings of the German Concrete Assn.

Deutscher Beton-Verein, Wiesbaden, V. I, 1939, V. II, 1949

Reviewed by PAUL ROGERS

These two volumes contain interesting and useful data on many phases of reinforced concrete engineering. It is interesting to note that the first volume was published at the peak of Nazi power and consequently is written in a tone of utmost arrogance. The second volume was published quite recently, and was written in a sober and scientific manner.

The first volume contains papers, not mentioning the strictly political ones, about the major reinforced concrete construction going on at that time. An extensive paper by Dischinger deals with the problem of creep in concrete. Another paper describes the numerous large-span thin-shell arch structures built outside Germany, including many in this country, claiming full credit for German engineers.

The second, post-war, volume contains an interesting paper by Morsch on the developments in reinforced concrete since 1900. In other papers numerous industrial structures, bridges, gravity and arch dams, viaducts, power stations, thin-shell roofs, locks, caissons, etc., are described. A paper by Enzweiler treats ready-mixed and transit-mixed concrete. Monumental, long-span structures are described by Dischinger in another paper, mentioning, among other things, Hitler's absurd idea to build an assembly hall with an arch roof of 750-ft span, to be built of brick masonry.

Fully 120 pages are devoted to a serious dissertation by many outstanding engineers to the development and practice of prestressed concrete. In another paper, Kurt Billig presents the application of lightweight concrete for hollow wall building construction in connection with housing projects in England. The last paper describes the details of a large dry dock in Hamburg.

Transfer of moments and deformations in concrete beams submitted to long-time loads (Momentomlagring och deformationer vid langtidsbelastade betongbalkar)

ARNE JOHNSON, *Betong* (Stockholm), V. 35, No. 3, 1950, pp. 231-249

AUTHOR'S SUMMARY

This paper, partly based on tests, deals with the effects produced by the most important factors on moment distribution and deformations in concrete beams submitted to long-time loads. In studying the effect of each individual factor, it is considered whether the part in tension is cracked or noncracked.

Concerning the moment distribution, the author arrives at the following conclusions:

At normal stresses, the plastic flow of concrete does not seem to exert any influence on the moment distribution.

As a rule, the shrinkage of concrete increases the moments above the supports both in cracked and in noncracked beams.

The formation of cracks usually gives rise to transfer of moments. The transfer of moments is dependent on the ratio of the amounts of reinforcement above the supports and between the supports. An increase in the amount of reinforcement above the supports causes an increase in the moments above the supports.

The change in the moments above the supports caused by shrinkage is, as a rule, greater than the corresponding change due

to formation of cracks. Therefore, the moments above the supports usually increase somewhat with the duration of load.

Regarding the deflections, the author's conclusions are:

The deflection is increased by the plastic flow of concrete, by the formation of cracks, and, as a rule, by shrinkage of concrete.

Within a relatively wide interval, the ratio of the amounts of reinforcement above the supports and between the supports exerts slight influence on the deflection so long as the yield point stress is not exceeded. This applies both to cracked and to noncracked beams. Consequently, an amount of reinforcement which differs to a relatively large extent from that calculated in accordance with the theory of elasticity usually will not cause any extra deflections.

The results obtained in this paper also ought to be applicable to structural members other than beams, *e.g.*, slabs.

The theory of deflections of solid web girders (Statik der Formaendarungen von Vollwandtragwerken)

LEOPOLD HERZKA, Springer-Verlag, Vienna, 232 pp. 130 Austrian Shillings ($16.00)

Reviewed by PAUL ROGERS

This book is devoted in its entirety to the difficult problem of deflections and deformations of solid web structural members. The author introduces and applies his own solution which he calls the Biegelinien-Polygon-Verfahren (BPV) or the Bending-line Polygon System. This system produces the deformations of solid web members by numerical solutions without the use of higher mathematics, although calculus was applied in the development of the basic formulas.

The author's new system is a development and extension of the elastic weights method. At any point, where the deformation is desired, an imaginary hinge is applied, and elastic weights as vertical loads and moments introduced. By an analogy, equations similar to the three-moment system are produced and solved simultaneously. This may suggest further simplifications by successive corrections.

The immense amount of information presented in this book can not be explained without some reference to the author's personal life. He was the leading bridge engineer of the Austrian Railways when Hitler's conquest of that country made him the member of an undesirable minority. Although he miraculously escaped physical destruction, he was, for many years, condemned to idleness, which he devoted to the preparation of this book.

The book deals extensively, and through many practical problems, with the deformations of beams with constant and variable moments of inertia, deflections due to torsional moments, deformations of cover-plated girders, Vierendeel girders, cantilevers and of vertical poles of variable moments of inertia. Eccentrically loaded columns of variable moments of inertia are extensiyely analyzed and buckling is thoroughly investigated. A separate chapter is devoted to rigid frames with straight and curved members of different shapes. The influence of tie-rod elongation is explained. A whole chapter deals with the problem of shear deflections and deformations due to nonuniform temperature change. Tables, charts and coefficients are presented and an extensive bibliography is included.

Exact results by moment distribution analysis (Exacte rekenwijze bij de methode "Cross")

J. C. LE NOBEL, *De Ingenieur* (Netherlands), Oct. 27, 1950, pp. 115-121

Reviewed by J. W. T. VAN ERP

The fact that the method of moment distribution since its presentation by Hardy Cross in 1929 has attained such widespread recognition in a short period of time proved its tremendous significance and practical value. The analysis of continuous frames thus became possible by simple arithmetical operations, and the designer of average schooling and dexterity could have the method at his command. The final result, however, remains an approximate one, albeit the degree of approximation can be carried out as close as desired down to the exact values. For most design problems in actual practice the result obtained after 4 or 5 cycles of reiterative moment distribution is accurate enough, seldom showing a difference of more than 5 percent from those obtained by exact analysis, the latter involving much more tedious algebraic manipulation. The author presents a method by which exact results can be obtained with hardly more

numerical ciphering than in conventional moment distribution. It involves the use of symbols until the very last stages of computation and thereby offers a better check than is possible in conventional moment distribution, being in general also more concise than the extensive numerical reiterations in the Cross method. The new method consists in finding the simple regularity which exists in the converging series of balancing moments, these series being geometric ones. To determine the characteristic of each series only two cycles of moment distribution have to be carried out and the exact final result can then be determined. Furthermore, the ratio of the various terms of the moment series for any of the joints of a specific structure consists mainly of the same element, thereby facilitating arithmetical work considerably. Translational moment distribution can also be done. The method is at its best for open structures, like continuous beams or portal frames, becoming more involved for closed and multistory frames, thereby losing its practical value. Nevertheless, the method should prove extremely valuable especially where exact results are imperative, avoiding involved algebraic operations, and for this reason the importance of the publication can hardly be overestimated.

Plates and shells (Flaechentragwerke)

KARL GIRKMANN, Springer, Vienna, 502 pp. 138 Austrian Shillings ($16.30)

Reviewed by PAUL ROGERS

This remarkable book is devoted to the theoretical analysis of structural members composed of plates, whether straight or curved in one or two planes. Much of its material has never been published in English and contains important and useful information to designers.

The first chapter introduces the reader into the general fundamentals of the mathematical theory of elasticity. Stresses in space are analyzed. Normal and shear stresses combined by the Mohr circle; Hooke's, Poisson's and Saint Venant's principles are explained.

One hundred and twenty seven pages are devoted to the problems of plates loaded in their plane (Scheiben). These include the elastic theory, Airy's stress functions, equations in polar coordinates, deformations, end conditions, approximate solutions, simplifications by Fourier's series, the theory of deep beams (diaphragms), approximate analysis by finite differences, the effect of concentrated loads, T-beam sections, circular ring forms, closed rings, stress concentrations, and numerous other problems.

The third chapter deals with the problem of plates. Equations for the normal, shear and torsional stresses are developed. After the general theories are fully explained, the problems of rectangular, triangular, circular and ring plates are analyzed. Continuous plates, flat slabs and similar forms are thoroughly investigated.

The fourth chapter treats the problems and theory of shells. After the introduction of the fundamental formulas, the membrane theory of rotation-shells is presented and its application to containers, tanks, and similar structures is explained. Tanks of equal stress condition, similar to the well-known Hortonspheres are presented and dome structures of different shapes are mentioned.

The theory of cylindrical shells is thoroughly explored. This includes the analysis of pipes as horizontal beams, the wind stresses on vertical cantilever vessels, the theory of Z-D shell roofs, the stress analysis of liquid-containing tanks of variable wall thicknesses, the cantilever action at the bottom of the tanks, the solutions of end conditions by iterations, and the stability of shells.

The fifth chapter presents the analysis of hipped-plate or pyramidical construction (Faltwerke). This excellent and very economical form of construction, which is especially adaptable to reinforced concrete, has a very slow start toward being adopted in this country, in spite of the excellent paper on the subject by Winter and Pei, (ACI JOURNAL, Jan. 1947). This kind of construction consists of a series of flat plates joined at angles and permitting the spanning of large areas without the use of beams and girders. In addition to the complete theory, a practical problem is also presented.

The last chapter deals with orthotrope plates, applicable to two-way solid and two-way joist slab construction.

This book is necessarily highly mathematical, and a working knowledge of calculus is essential to the understanding of the theories presented.